CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

JUNE, 1953. **Vol. XLVIII, No. 6**

FORTY-EIGHTH YEAR OF PUBLICATION

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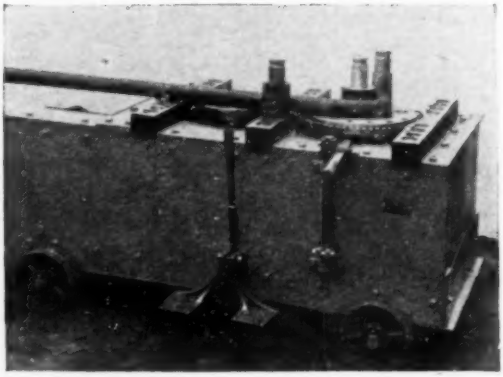

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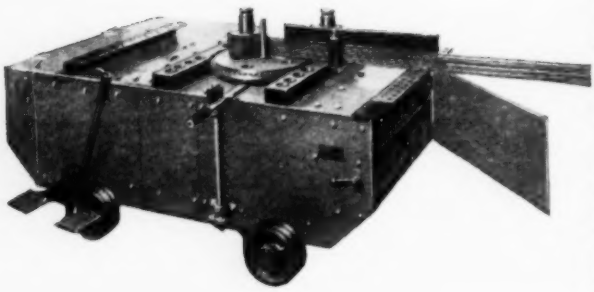

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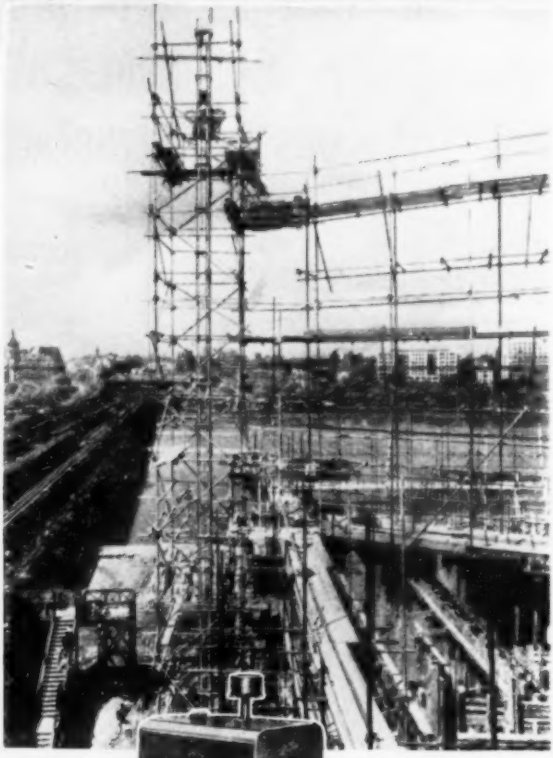

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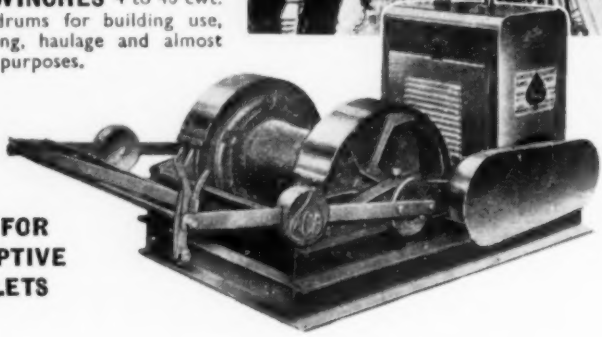

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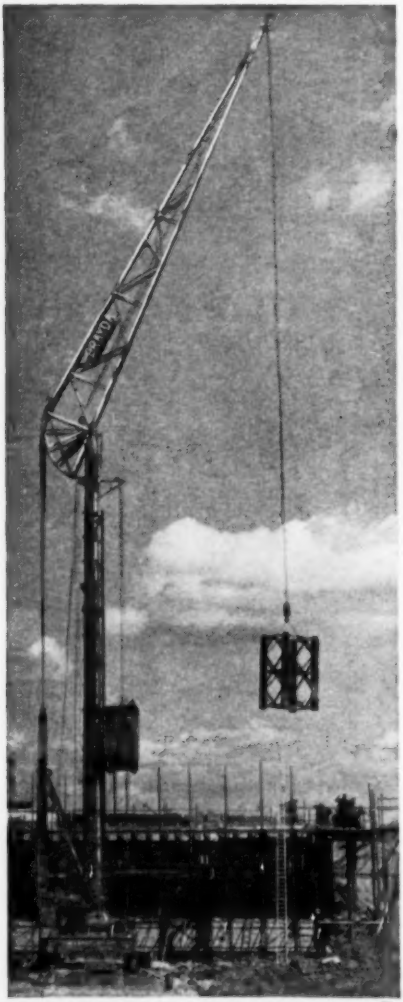

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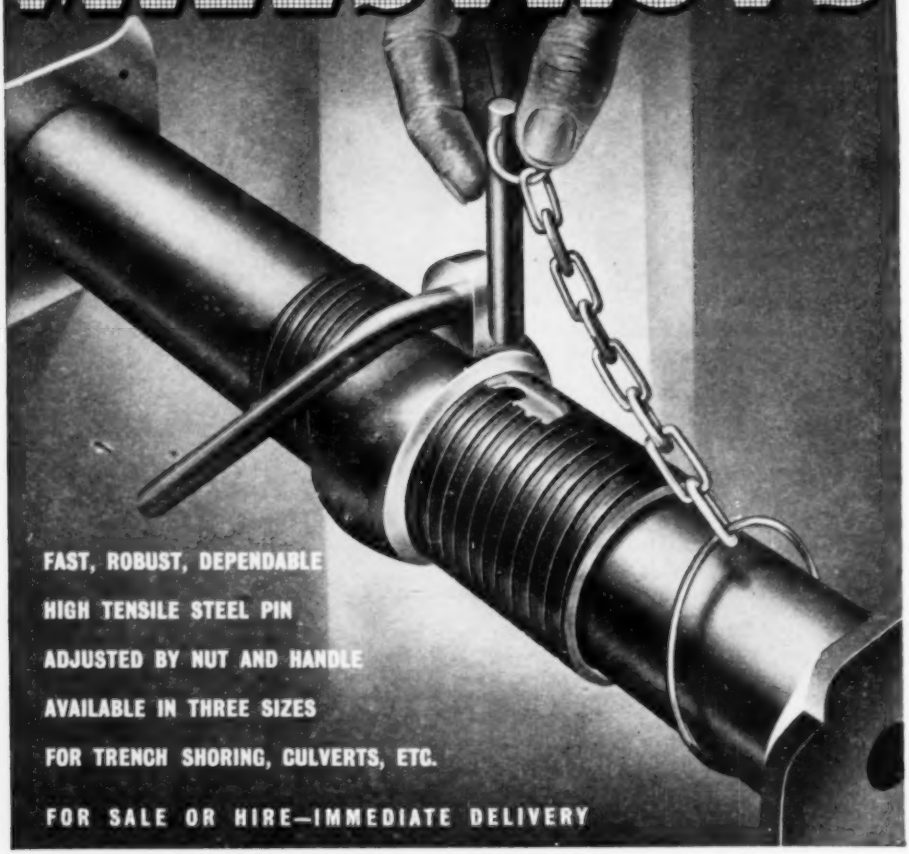

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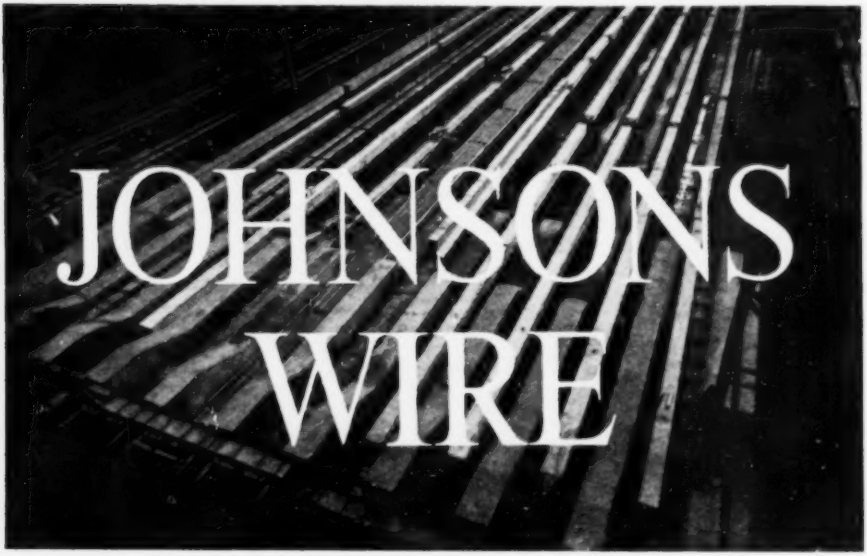

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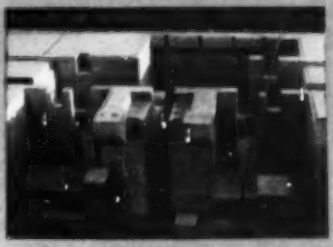

This intricate foundation was constructed for the Dumbarton Gas Works in Scotland.

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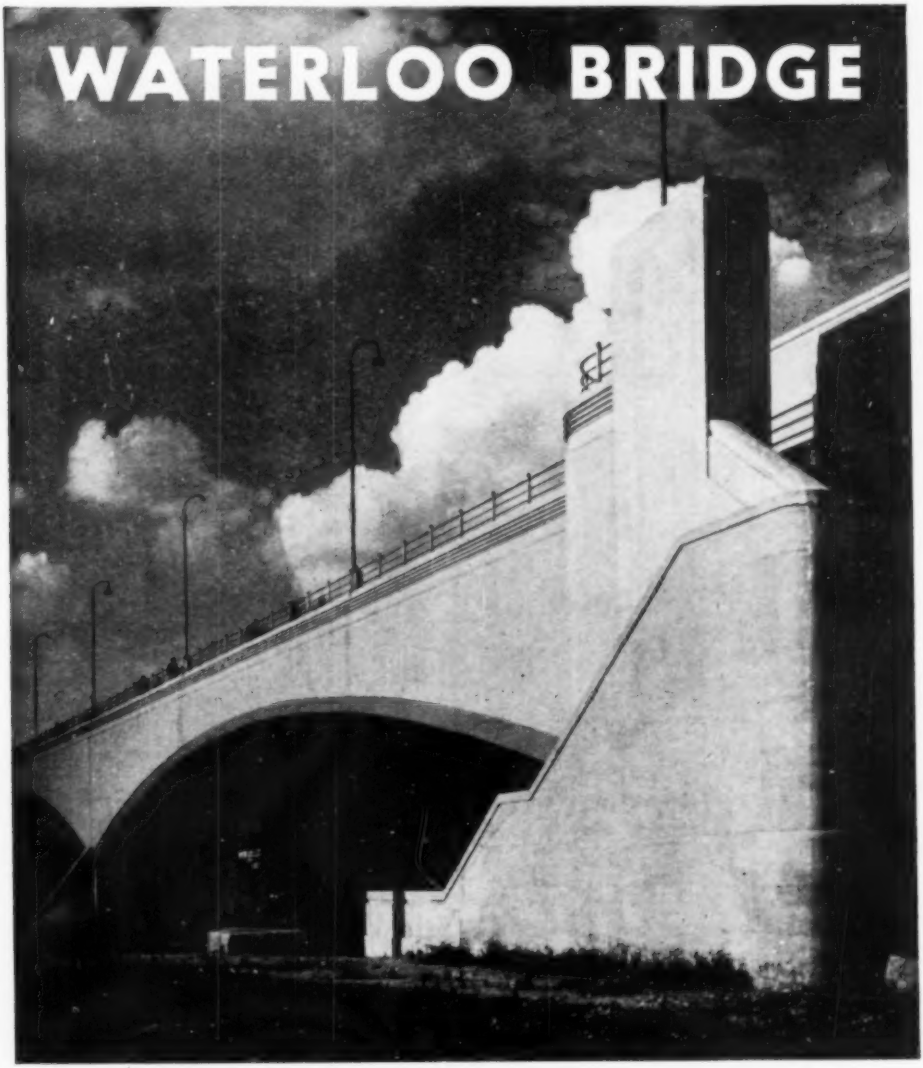

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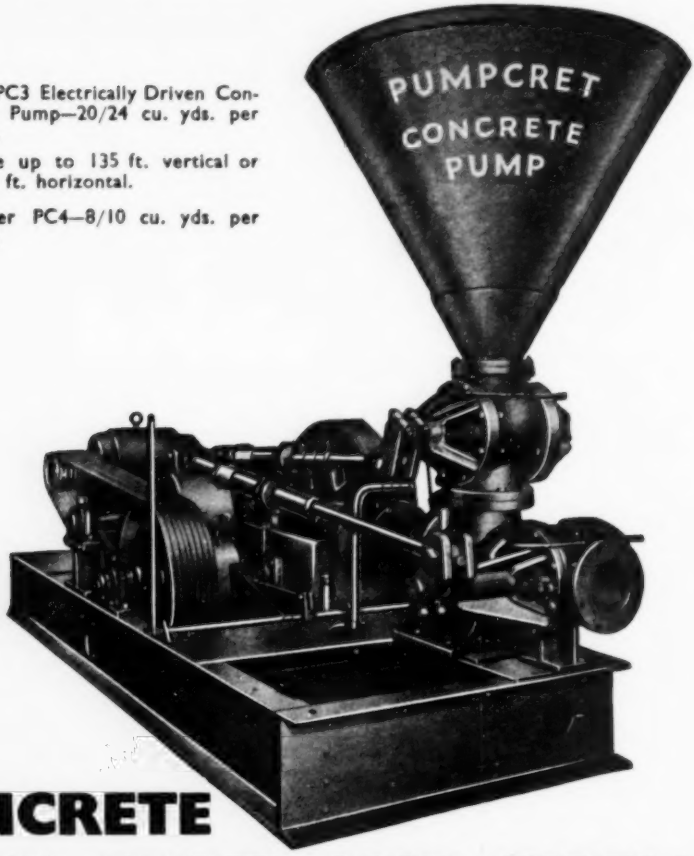

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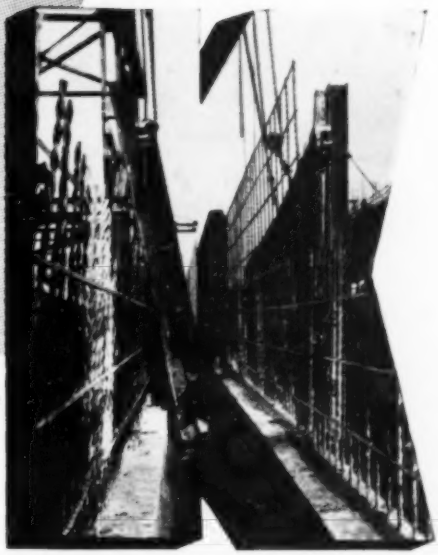

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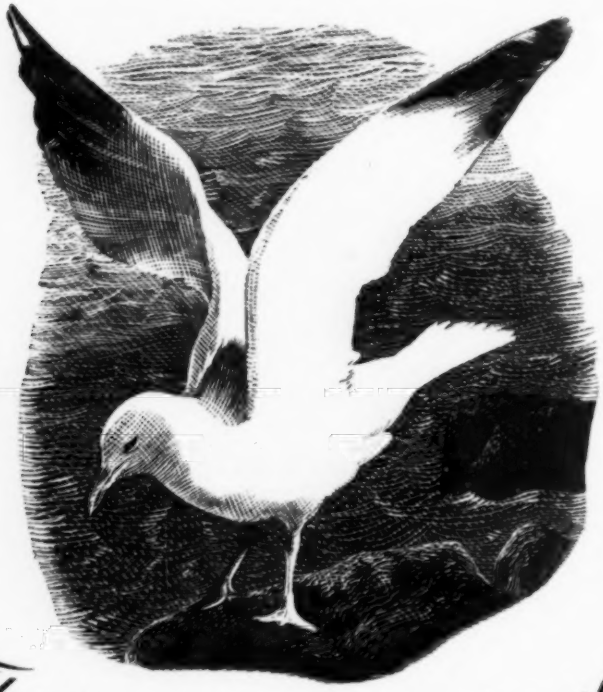

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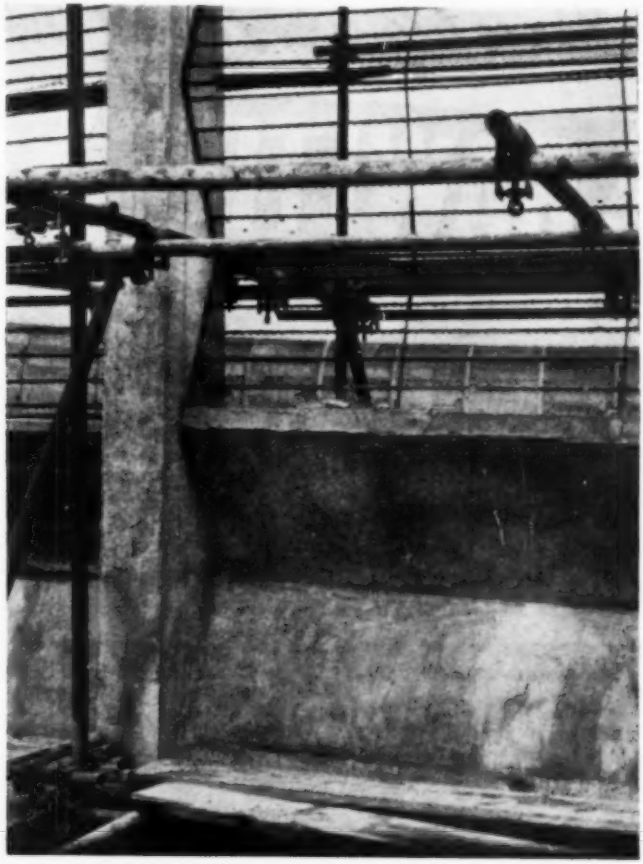

* *Designed by Indented Bar & Concrete Engineering Co., Ltd.*

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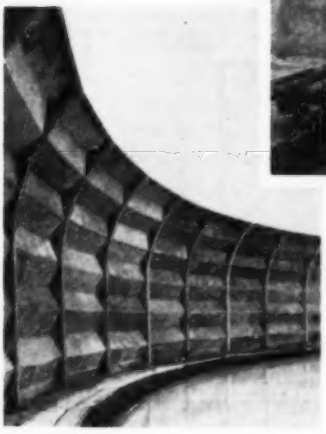

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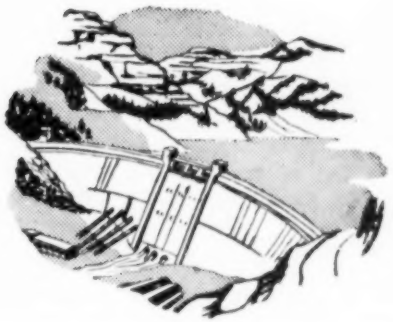

The Cementation process is now being extensively used in sealing off leakages of water from large tanks, docks and other similar structures, and in rendering watertight tunnel linings. Contracts of this nature have been carried out for the South Metropolitan Gas Works, the Gas Light and Coke Co., the Meltham U.D.C. where escapes of water were occurring from the gasholder tanks, at the Burry Port Dock of the G.W.R. Barry Dock, and Leeds Electricity Works Canal, where loss of water through the walls and floors of the structure was becoming serious. Similarly, cementation has been employed in rendering watertight the swimming baths at Clifton, Bristol, and Bradford-on-Avon, and the sewage disposal tanks for the Doncaster Corporation. It has also been used for waterproofing of basement walls and floors as at the new B.B.C. House, London, Stockport Sewage Works, Grimsby Power Station, and for many other purposes.

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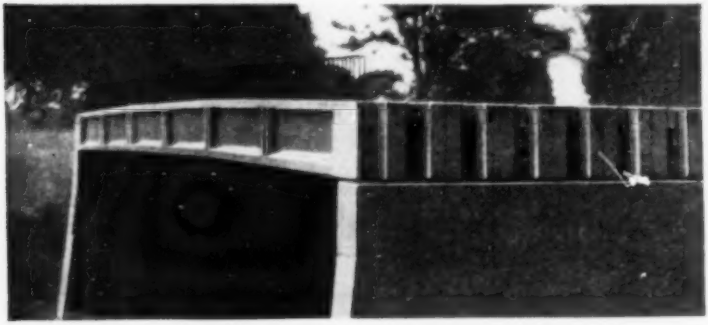

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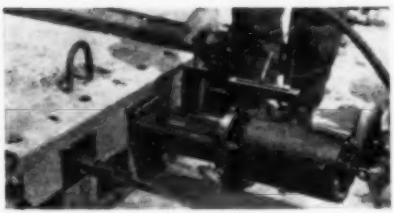

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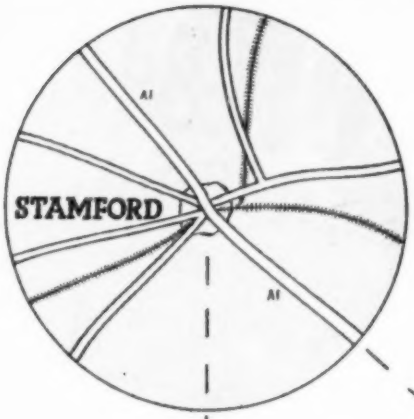

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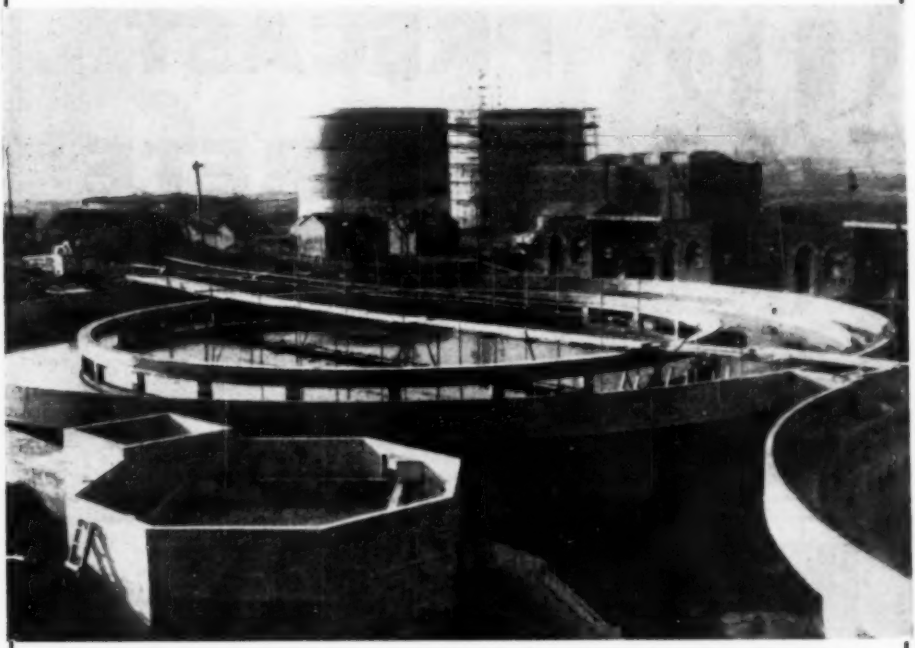

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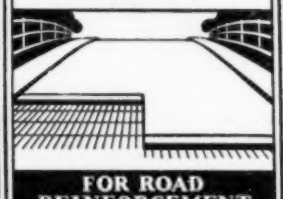

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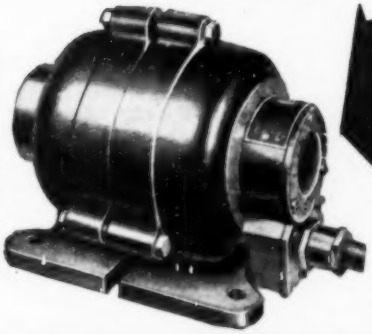

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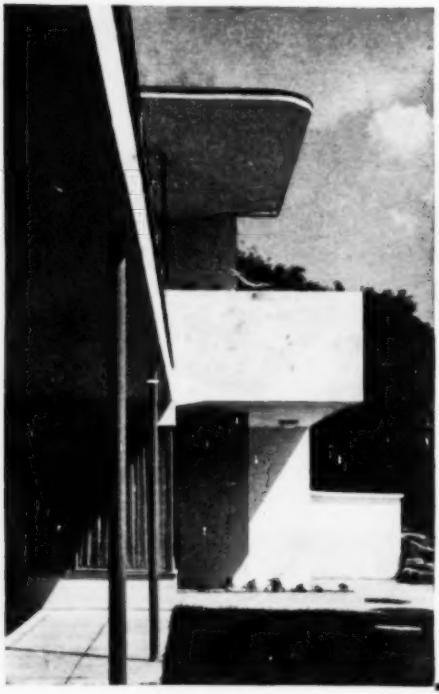

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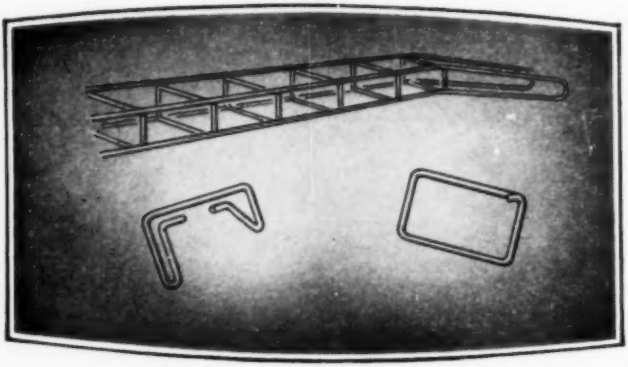

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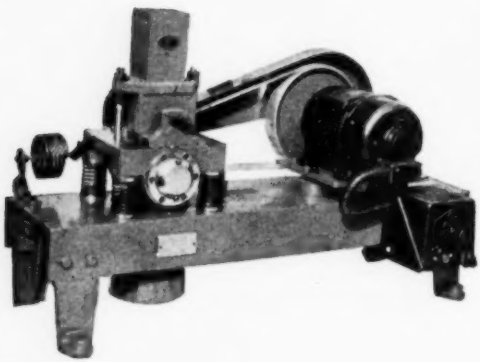

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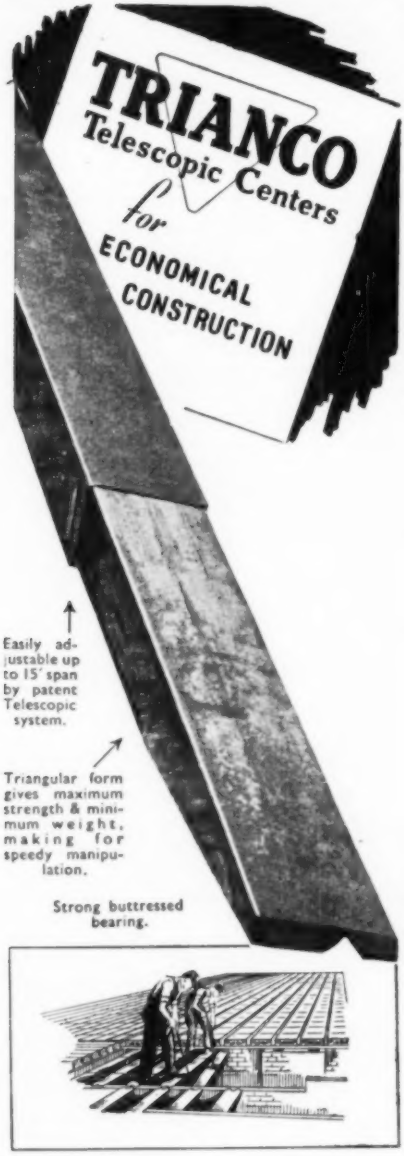
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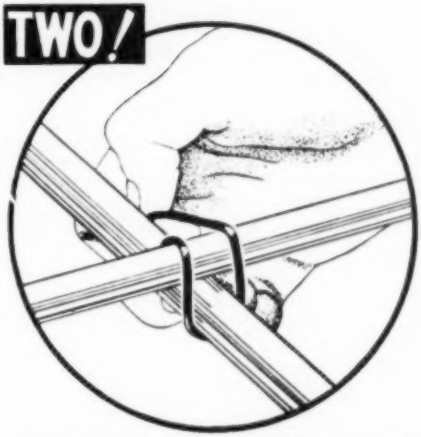

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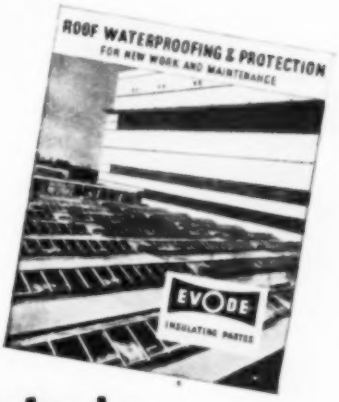

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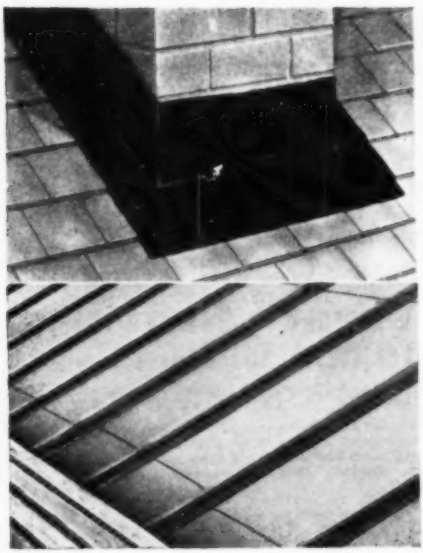

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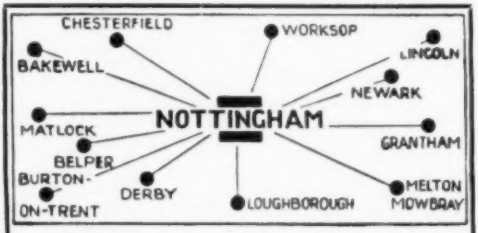

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The photograph shows a section of the roads recently completed by the Yiewsley and West Drayton U.D.C. on the Philpots Farm Housing Estate, West Drayton. The roads were constructed by *stabilising the naturally occurring brick earth and hoggin* to a depth of 6" with 10% by weight of cement, employing the "mix-in-place" method. The soil-cement, surface dressed with tar and shingle, will carry all the builders' traffic. The method is both speedy and economical, 12,000 square yards having been completed in 15 working days at an approximate cost of 7/- per square yard. Kerbs and a 1" gravel asphalt carpet will be added upon completion of the building operations.

Observe the SISALKRAFT Prefabricated Curing Blankets used throughout for the curing and protection of these roads.

Photograph by kind permission of the Yiewsley and West Drayton U.D.C. (Engineer, Surveyor and Architect, W. T. Morgan, F.R.I.C.S., M.I.Mun.E., L.R.I.B.A.), Town Hall, West Drayton, Middlesex.

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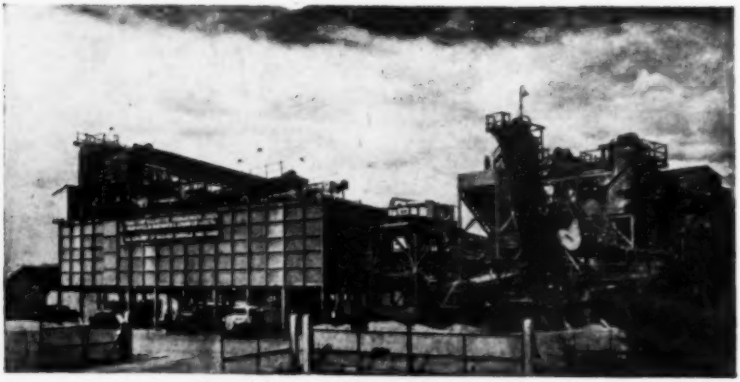

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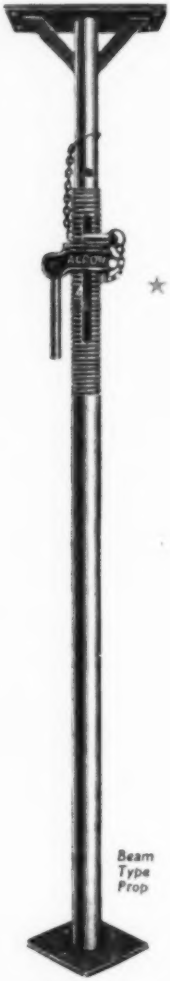

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PROP SIZE No.	HEIGHT		WEIGHT
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1	5 ft. 7 in.	9 ft. 10 in.	48 lb.
2	6 ft. 7 in.	10 ft. 10 in.	50 lb.
3	8 ft. 2½ in.	12 ft. 5½ in.	58 lb.
4	10 ft. 6 in.	15 ft. 6 in.	78 lb.

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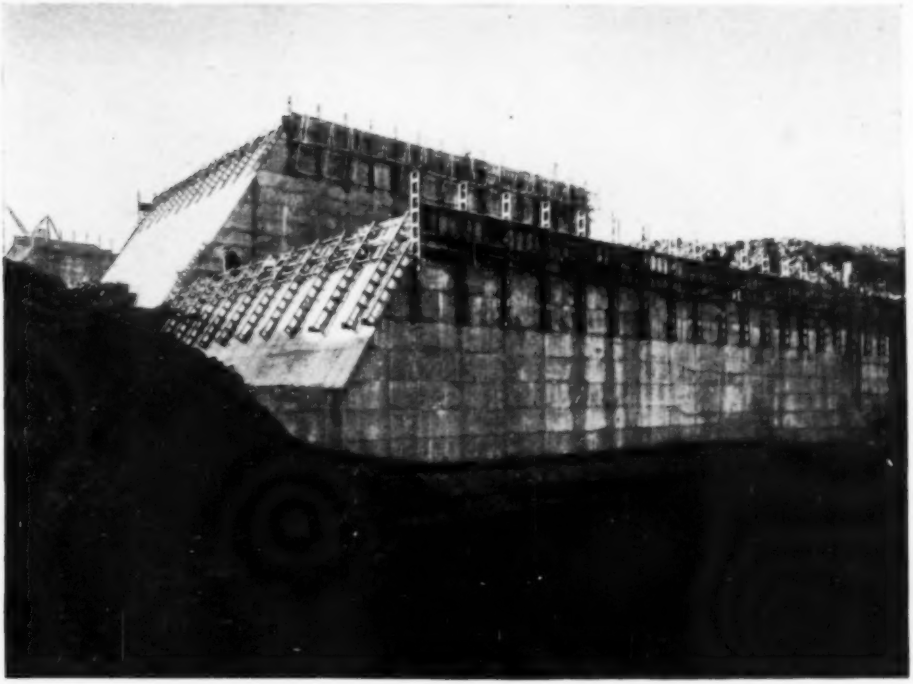

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Concreting in progress on one of the 24 monoliths which together form the mass concrete gravity type dam. The contract for the Fife County Council covers the construction of a mass concrete impounding dam, 160 ft. high × 1,300 ft. long, requiring 160,000 cubic yards of concrete. This dam on the river Devon will supplement the storage capacity of an existing reservoir by 1,200,000,000 gallons.

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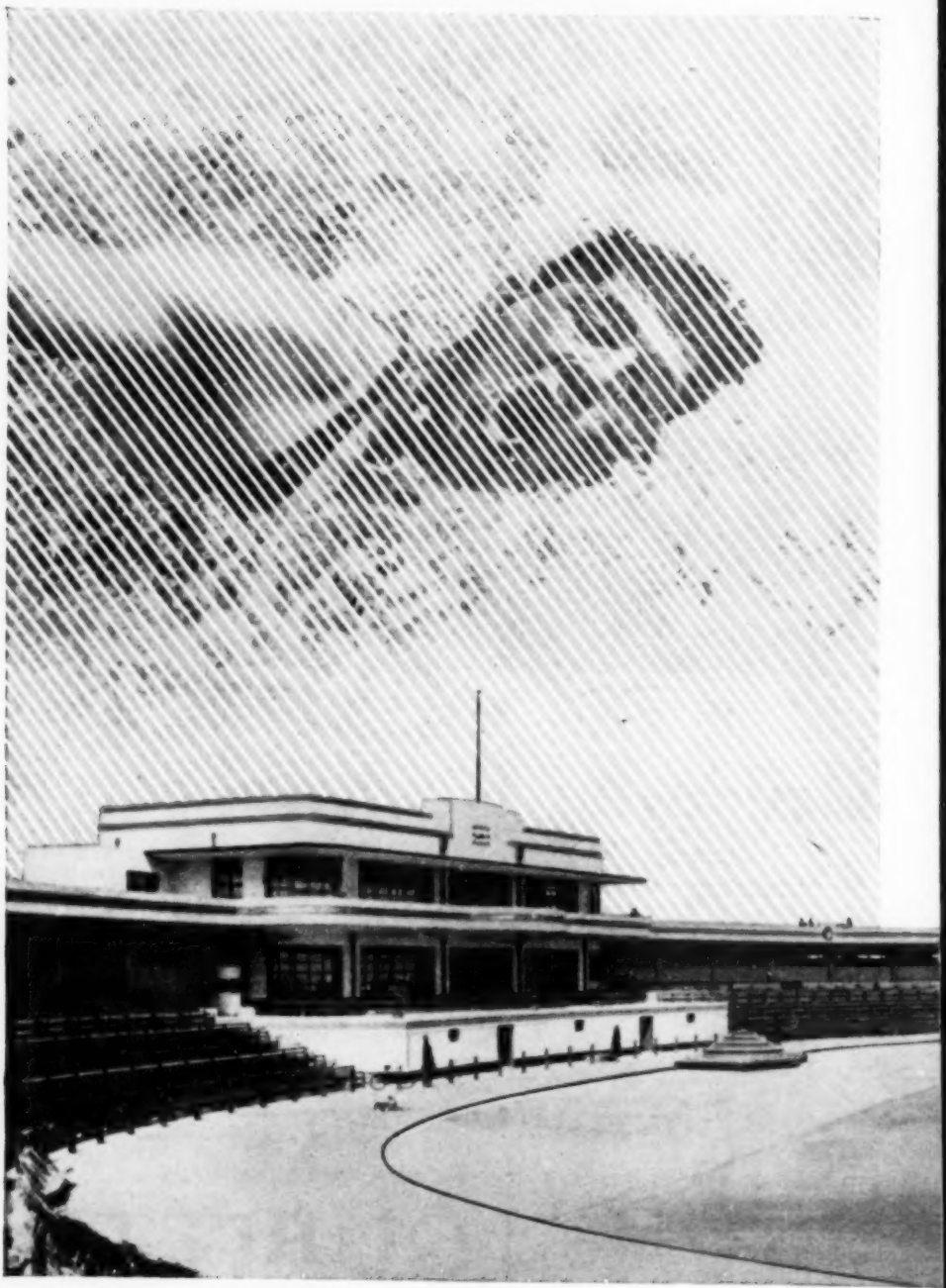

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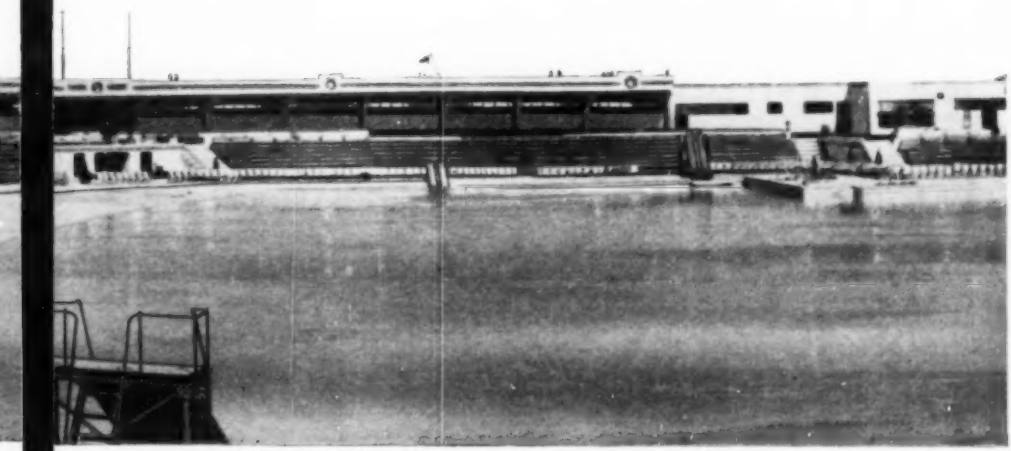

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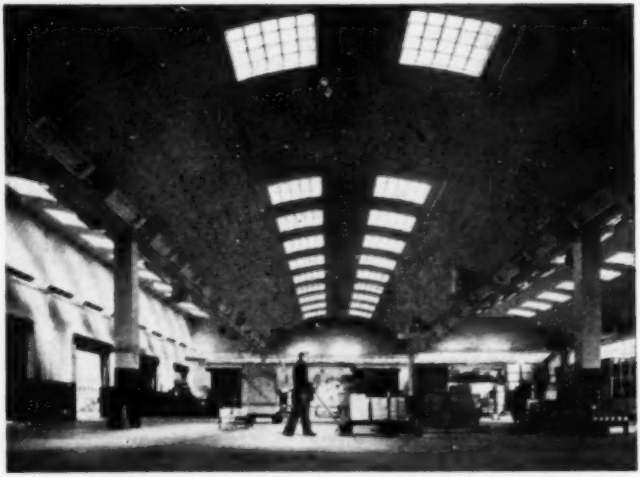

Factory, London. *Architects*: Messrs. Clifford Tee & Gale.

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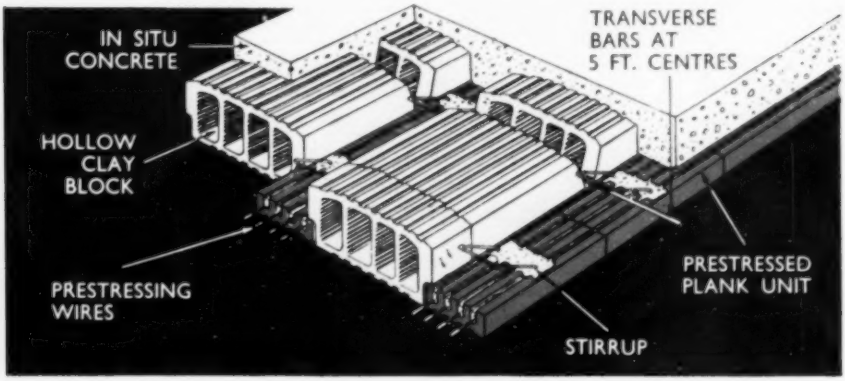

Stahlton Prestressed Floors

STAHLTON prestressed clay block floors and roofs embody the combined advantages of prestressing, light weight, fire resistance, adaptability and ease of erection. The principal component is a factory made prestressed earthenware plank containing high-tensile steel wires embedded in vibrated mortar in grooves in the upper surface. Hollow clay filler blocks are placed between the planks giving a uniform clay soffit which is ideal for the application of plastered finishes. The floor is completed by placing *in-situ* concrete to the required thickness.

Tests show that Stahlton has a very high degree of fire resistance and in this respect it is superior to other types of flooring.

With a range of hollow-block sizes, from 4″ to 8½″ deep, floors can be constructed to suit all types of loading with spans up to 30 ft. and more.

Transverse distribution of loading is provided by reinforcement placed at approx. 5-ft. centres across the prestressed planks.

No shuttering is required. The planks are temporarily supported at approx. 5-ft. centres during construction. These supports may be removed when the *in-situ* concrete has reached an ultimate strength of 2,500 lb. per sq. in., normally 7 to 10 days.

The carrying capacity of a floor of any given thickness can be increased under heavy concentrated loads, such as partitions, by placing two or more planks side by side and omitting the infilling hollow blocks.

The hollow blocks may be cut to allow for electric light drops, sprinkler nozzles, etc.; conduit and pipework for which can be accommodated in the thickness of the *in-situ* concrete.

Openings for stairways, roof lights and large pipe ducts, etc., can be formed by trimming with two or more planks placed side by side or *in-situ* concrete beams within the depth of the floor.

Stahlton floors can be readily erected by the contractor on the site.

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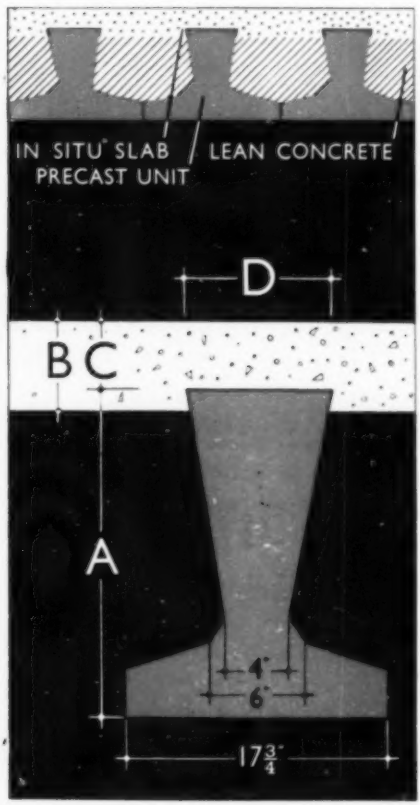
Prestressed Concrete Main Bridge Beams

designed for
Ministry of Transport Loading
Series WR



Section No.	Dimensions A	B	C	D	Wires 0·2" h.t.	Span feet	Total Weight T.	C.	Q.	Lb.
WR1A	12"	2½"	2"	6·7"	16	16	0	15	1	4
,,	,,	,,	,,	,,	19	18	0	17	0	22
,,	,,	,,	,,	,,	23	20	0	19	0	12
,,	,,	,,	,,	,,	27	22	1	1	0	2
,,	,,	,,	,,	,,	31	24	1	2	3	20
,,	,,	,,	,,	,,	35	26	1	4	3	10
WR1B	12"	3½"	2½"	6·7"	16	16	0	15	1	4
,,	,,	,,	,,	,,	19	18	0	17	0	22
,,	,,	,,	,,	,,	22	20	0	19	0	12
,,	,,	,,	,,	,,	26	22	1	1	0	2
,,	,,	,,	,,	,,	30	24	1	2	3	20
,,	,,	,,	,,	,,	34	26	1	4	3	10
,,	,,	,,	,,	,,	37	27	1	5	3	5
WR2A	14"	3½"	2½"	7·3"	32	26	1	9	3	22
,,	,,	,,	,,	,,	36	28	1	12	1	0
,,	,,	,,	,,	,,	41	30	1	14	2	6
WR2B	14"	4½"	3½"	7·3"	31	26	1	9	3	22
,,	,,	,,	,,	,,	35	28	1	12	1	0
,,	,,	,,	,,	,,	40	30	1	14	2	6
,,	,,	,,	,,	,,	42	31	1	15	2	23
WR3A	16"	4"	3"	8"	37	30	1	18	1	6
,,	,,	,,	,,	,,	42	32	2	0	3	12
,,	,,	,,	,,	,,	47	34	2	3	1	18
WR3B	16"	5"	4"	8"	37	30	1	18	1	6
,,	,,	,,	,,	,,	41	32	2	0	3	12
,,	,,	,,	,,	,,	46	34	2	3	1	18
,,	,,	,,	,,	,,	48	35	2	4	2	21
WR4A	19"	5"	3½"	9"	43	34	2	13	2	26
,,	,,	,,	,,	,,	47	36	2	16	3	16
,,	,,	,,	,,	,,	52	38	3	0	0	6
,,	,,	,,	,,	,,	57	40	3	3	0	24
WR4B	19"	6"	4½"	9"	42	34	2	13	2	26
,,	,,	,,	,,	,,	47	36	2	16	3	16
,,	,,	,,	,,	,,	52	38	3	0	0	6
,,	,,	,,	,,	,,	57	40	3	3	0	24
WR5A	22"	5"	3½"	10"	50	38	3	11	2	10
,,	,,	,,	,,	,,	55	40	3	15	1	12
,,	,,	,,	,,	,,	60	42	3	19	0	14
,,	,,	,,	,,	,,	65	44	4	2	3	16
,,	,,	,,	,,	,,	66	46	4	6	2	18
,,	,,	,,	,,	,,	66	48	4	10	1	20
WR5B	22"	6"	4½"	10"	49	38	3	11	2	10
,,	,,	,,	,,	,,	54	40	3	15	1	12
,,	,,	,,	,,	,,	59	42	3	19	0	14
,,	,,	,,	,,	,,	64	44	4	2	3	16
,,	,,	,,	,,	,,	66	46	4	6	2	18
,,	,,	,,	,,	,,	66	48	4	10	1	20

COSTAIN CONCRETE CO. LTD

Prestressed piling by Costain Concrete Co. Ltd.

Prestressed bridge beams and prestressed piles may be seen in the illustration above, which shows a production bay at the Coltness Factory of Costain Concrete. Prestressed piles by Costain were used to form the retaining wall for the Singer Manufacturing Co.'s Factory on the Forth and Clyde canal, illustrated below.

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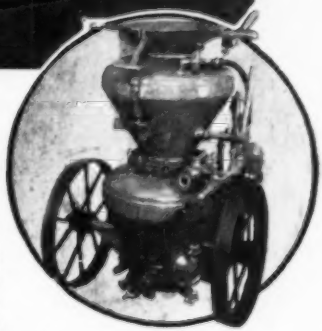

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Volume XLVIII, No. 6. LONDON, JUNE, 1953.

EDITORIAL NOTES

The Education of Technologists.

THE problem of what should be included in the education of technologists has frequently been discussed, and it is a measure of its importance that the subject should be so frequently brought forward. Recently Sir Edward Salisbury, F.R.S., Director of the Royal Botanic Gardens at Kew, speaking at the opening of the new biological laboratories at the University College of the South West, said—"There is a temptation to put more and more facts into a syllabus, but, though a limited equipment of facts is a necessary part of training, the ideas that are illumined and deduced from the facts are much more important. The tendency to regard technical equipment as an end in itself is too prevalent in schools, and its correction in the universities is therefore the more important. Two things are essential to a liberal education—breadth of sympathy and mental humility. It is all too easy for specialists to delve deeper into one particular furrow, and so become isolated by the by-products of their own activities, perhaps in the form of a superabundant technical jargon . . . even to the extent of being unable to convey their thoughts in good English." With these words few will disagree, except that some of us might describe the result of education as knowledge rather than equipment. But on the question of how to remove the cause of the criticism there are different views. There is, however, increasing agreement that the ideal solution is one that has been advocated in this journal for many years, namely that candidates for an engineering degree should have made a study of the arts.

A leading article in "The Times" discussing Sir Edward Salisbury's speech suggests that the solution lies in the avoidance of premature specialisation in the last years at school and that a youth who has been given a taste for arts, literature, and history will have no need for compulsory lectures in these subjects while attending a course on a technical subject at a university, for they will have become part of his life and will be the companions of his leisure. This is the view that we have for long advocated, for it does not seem possible that arts subjects can be dealt with usefully in the course of a technical training. An improvement in the general education of students of technical subjects, and of their ability to write good English and to spell correctly, could probably be obtained if papers were assessed on their presentation in plain language and with a logical development of the subject as well as on the technical accuracy of their contents. Examiners who pass candidates whose papers contain correct facts and ideas expressed in an execrable manner are hardly to blame in the

present circumstances when so few candidates seem able to express themselves in good English or to spell correctly, and when examiners are concerned only with the technical knowledge of the candidates. Indeed, the education of some lecturers and examiners may have been similar to that of some of the students, and their own general knowledge equally weak. There has, in fact, grown up a technical jargon in which lecturers give to words the opposite of their dictionary meaning. The correct use of one's own language is surely the most important subject at all stages of education, and its teaching should not be left until the age at which technical training starts. To obtain the greatest benefit from a technical course it is necessary that a student should be able to set down clearly what he has understood at his lectures, and the ability to do this should be his before he attends technical courses.

It is common at present for a youth to leave school at the age of sixteen or so, and then to concentrate on a technical subject. As the possessor of a school certificate, he generally appears to consider that his education is complete, and that it only remains to master a technical subject for him to be a full man able to earn a living. The same problem has been encountered in the U.S.A., and as a result some technological colleges have made arrangements by which candidates will receive a more liberal education before they are accepted for technical training. In these cases the candidates must attend a college where the liberal arts are taught for a period of two to three years, and will then attend the technical college for two years. This arrangement has much to commend it. The total period of attendance at a university or college is not increased, and it is thought that a man with a background of two years of study of the arts, which would include mathematics, will be able to assimilate technical knowledge in a shorter time than a younger man with no education beyond attendance at school up to the age of sixteen or so. In this the American technological colleges are following the lead of Dublin University where it has always been necessary for a student at the engineering college to have a pass degree in arts before he can obtain a degree in engineering. We have never heard a graduate of this engineering school complain that time spent in studying arts subjects was wasted ; it is certainly not true that graduates of the engineering college at Dublin University have less knowledge of engineering subjects than graduates of other universities. The fact that a study of the arts at this university is a prerequisite of an engineering degree is the answer to those who claim that it takes so long to learn engineering that there is no time to study the arts unless the time spent at a university is to be unreasonably long. The American experiment on the same lines will be followed with much interest, and it would be useful if a similar experiment were to be made in Great Britain. In the year 1949 the then Parliamentary Secretary of the Ministry of Education, who had himself been a schoolmaster, said that specialisation at schools tended to produce illiterate, if ardent, specialists, who entered the universities with only a mediocre capacity to speak and write their mother tongue, and with a detachment from living things that was the death-knell of a vigorous cultivated mind. Changes of Government have brought changes of Ministers and Parliamentary Secretaries, but it is encouraging to know that the narrow outlook that is a common result of specialisation, and the difference between education and technological training, have been put on record by such a high authority.

A Large Factory for Prestressed Concrete Products.

THE factory at Newmains, near Glasgow, of the Costain Concrete Co., Ltd., is stated to be the largest factory in the world producing prestressed precast concrete products on a long-line process. The casting shop, which covers an area of 600 ft. by 500 ft., contains 112 stressing beds 470 ft. long arranged in 56 pairs, and a number of shorter beds from 20 ft. to 125 ft. long. A view looking along one of the six bays into which the shop is divided is shown in *Fig.* 1. Separate buildings accommodate plant maintenance, mould manufacture, quality control and research laboratory, drawing offices for design and production planning, general administrative offices, canteen, etc. Although the factory is highly mechanised to reduce the labour required, at present between 300 and 350 men are employed.

Structural members of any design can be made up to a total prestressing force of 200 tons per unit. Standard products include sheet piles, bearing piles, lighting columns, transmission-line poles, bridge beams up to 50 ft. span, a patented floor recently introduced into this country from Switzerland, and " E " class sleepers for British Railways. In addition to pre-tensioned bonded units, post-tensioned units are also made when required.

Piles up to 75 ft. long are cast on the long beds, whereas members such as bridge beams and poles are cast on the shorter beds. A form of transmission-line pole designed as a Vierendeel girder is shown in *Fig.* 2 on a bed after the mould was removed.

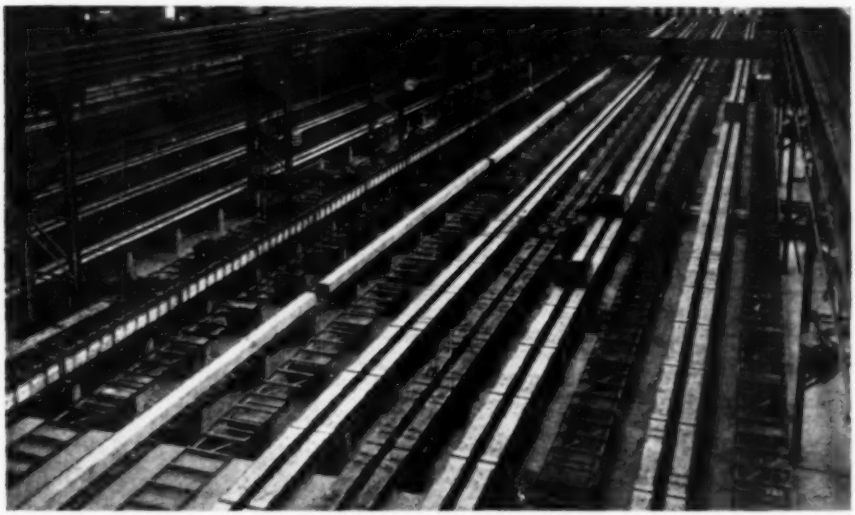


Fig. 1.—One Bay of the Casting Shop.

The standard bridge beams are of the cross section shown in *Fig.* 3 and the same mould can be adapted to cast them in depths varying from 1 ft. to 1 ft. 10 in. The smaller beams, containing sixteen 0·2-in. diameter high-tensile wires, have been approved for Ministry of Transport loading for spans up to 15 ft. when used in conjunction with an in-situ topping 2½ in. thick. The larger beams are stressed with up to 70 wires and have an in-situ covering 6 in. thick for spans of about 50 ft. Stirrups are used to bond the beams to the in-situ slab, and the tops of the larger beams are castellated.

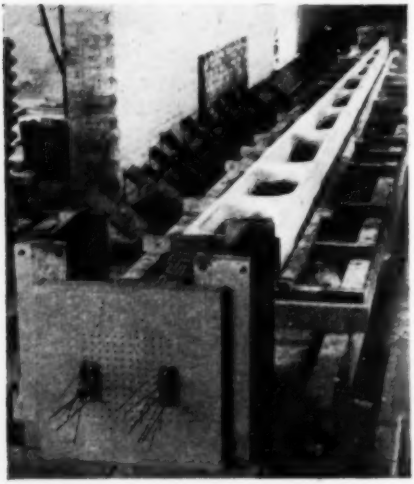


Fig. 2.—Transmission-line Pole.

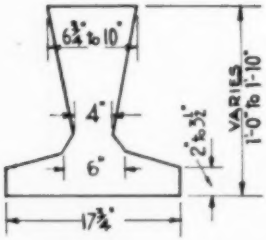


Fig. 3.—Cross Section through Bridge Beam.

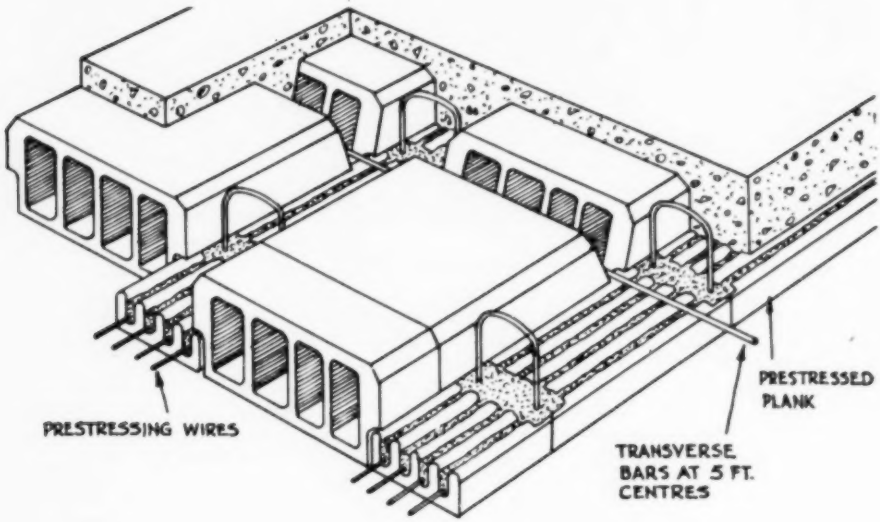


Fig. 4.—The "Stahlton" Floor.

The prestressing wires used in the products are 0·2 in. diameter or 12 gauge (mostly the former), and plain and indented wires are used. For use on the longer beds the wire is supplied in coils of 2 ft. diameter, and for shorter members and post-tensioned work in 8-ft. coils. Before it is used any grease or other coating which might impair the bond is removed from the steel by washing it first in a hot detergent solution, then in acid, and finally rinsing the wire in hot water.

grooves are filled with fine concrete. The prestressed planks are used as a combined reinforcement and shuttering to support hollow clay tiles (*Fig.* 4), so that only the concrete in the topping and part of the ribs has to be placed on the site.

The projecting stirrups ensure that the planks and the in-situ concrete act as a composite member. The planks are temporarily propped at 5-ft. centres during construction. The in-situ concrete topping varies from $\frac{1}{2}$ in. to 2 in. in thickness, and services such as electrical conduits are

Fig. 5.—"Stahlton" Floor Ready for Concreting.

Floors of Prestressed Planks and Hollow Tiles.

The beds 470 ft. long are used for making units for the patented "Stahlton" floor. This comprises clay tiles $5\frac{5}{8}$ in. wide by 12 in. long by $2\frac{3}{8}$ in. thick with four grooves $1\frac{5}{8}$ in. deep formed in their upper surface (*Fig.* 4). The tiles are placed about $\frac{1}{4}$ in. apart on the beds to form a plank one tile wide and of the length required. The tiles are separated from one another by precast concrete spacers $\frac{1}{2}$ in. thick. High-tensile wires are laid in the bottoms of the grooves and tensioned. Rich Portland cement grout is then placed and vibrated into the grooves and the spaces between the tiles, stirrups are fixed as required, and the normally cast in this topping. The hollow tiles are varied in depth and width in order to produce a combination of prestressed planks and depth of floor suitable for various loads and spans up to 30 ft. Where partitions are to be supported, two or more planks may be placed side by side in order to form a stronger rib. The planks may also be used as lintels, in which case they are built directly into the brickwork; their depth of $2\frac{3}{8}$ in. is equal to one course of bricks. A floor ready for concreting is shown in *Fig.* 5. This type of floor is being used in large contracts for the London County Council, the County Borough of Southampton, the Scottish Special Housing Association, Ltd., and in the construction of multiple-story residential flats at Clydebank. It is

Fig. 6.—Threading High-tensile Wires through Fixed Anchor-plates.

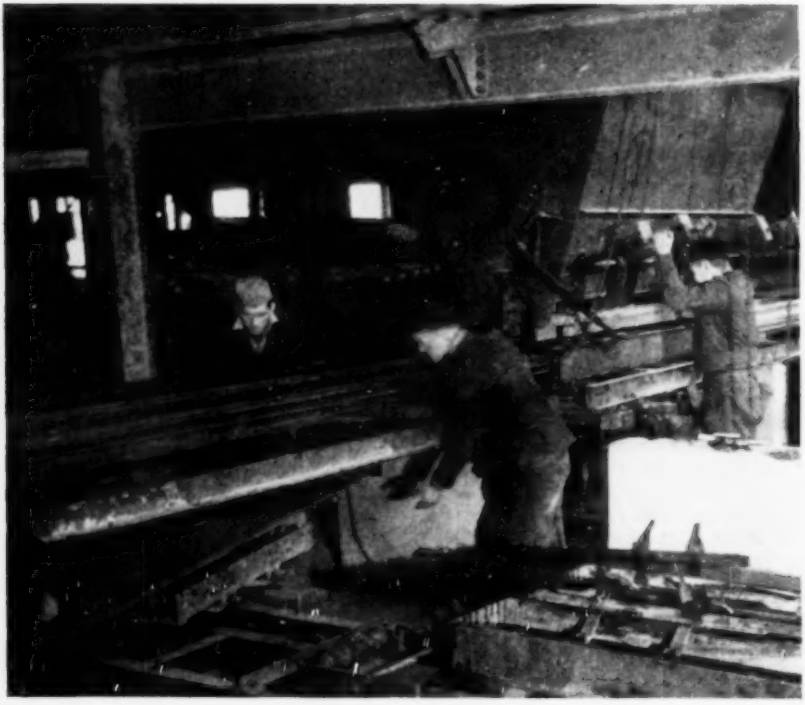


Fig. 7.—Moulds for Sleepers being Raised on to Stressing Beds.

also being extensively used in schools and hospitals.

Railway Sleepers.

The larger part of the production of the factory consists of railway sleepers. These are being made at a rate of 300,000 per annum, and the potential capacity is 750,000. The sleepers are also made on the long beds.

Each stressing bed is in the form of a roller conveyor, the steel rollers being supported at working height on steel angles spanning between concrete blocks. Wires are tensioned two at a time by hydraulic jacks, or singly by an electric winch-pulley combination with an electrical stopping and starting control. The tensioning machines, of which there are several, are arranged to traverse the entire width of the shop (*Fig.* 8). A sliding jaw connects the wires to the jacks. At the opposite ends of the beds the wires are anchored by cylindrical steel grips bearing against anchor-plates which transfer the load through distributing beams to the abutments. A similar anchorage is used at the jacking end of the beds. The wires are fed from spool-trucks which are arranged so that they can travel on a track across the entire width of the factory (*Fig.* 6). The wires are mechanically pulled twenty at a time from one abutment to the other by means of an electric winch.

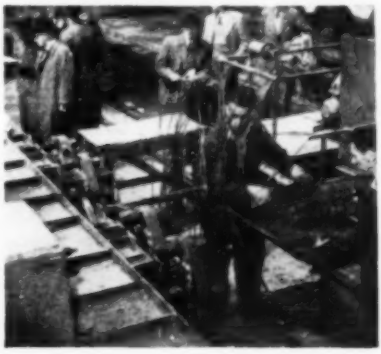


Fig. 8.—Tensioning Wires on a Long Bed.

The cleaned and oiled moulds travel under the stressing beds on conveyors extending across the width of the factory (*Fig.* 7). These conveyors are placed centrally across the shop in line with the overhead crane from which the concrete is distributed. The moulds automatically stop at the stressing bed in use, and are raised on a rotatable frame, turned through 90 degrees, and placed on the bed so that the wires pass through the slotted end-plates of the moulds. Metal

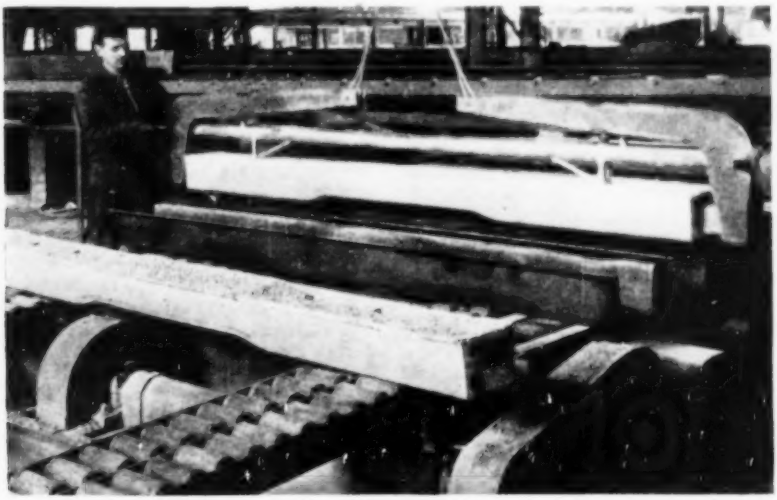


Fig. 9.—Lifting Sleepers from Moulds.

cross-straps and pins for forming bolt holes for fixing the chairs are clamped to the moulds. When the clamps are in position the moulds are pulled along the bed to a vibrating table, 10 ft. by 3 ft., under the filling hopper (*Fig.* 7) from which the concrete is discharged while the moulds are vibrated. The filled moulds are pulled along the bed to come to rest adjacent to the previously-filled mould. Here the concrete is further compacted by a heavy-duty vibrator working at 8500 revolutions per minute, which is attached to the

cally conveyed to a position outside the stressing-bed area, where the sleepers are removed from the moulds by the device shown in *Fig.* 9 and deposited on a gravity roller-conveyor which delivers them to a platform where the metal chairs are attached (*Fig.* 10). The sleepers are checked for gauge and other dimensions prior to being hoisted by overhead crane into railway trucks on an adjacent track. The moulds are cleaned and oiled, and delivered by conveyor to the casting position. At each of the positions where

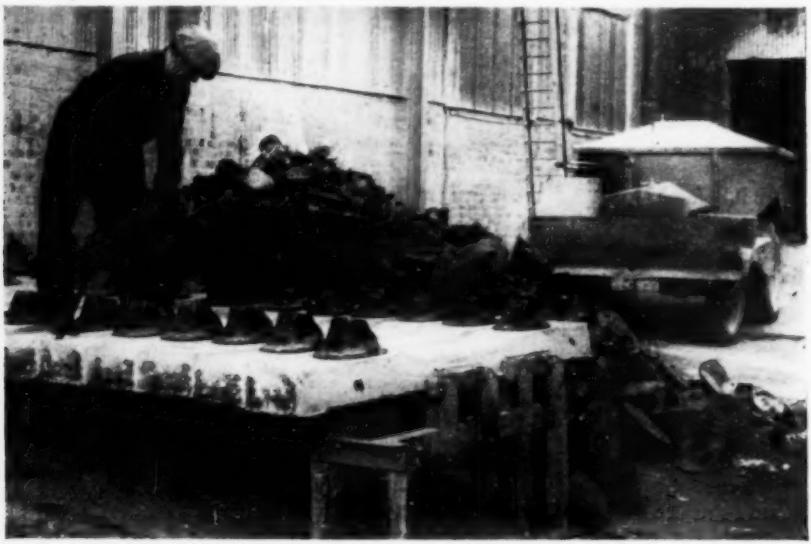


Fig. 10.—Fixing Chairs to Sleepers.

mould. The top surface of the concrete, which is the bottom of the finished sleeper, is screeded to a rough finish with trowels. The cycle of operations is based on making a sleeper every three minutes, and this has been maintained over a number of years.

At present the moulds are left to cure on the beds for five days before the anchors are removed from the ends of the wire. Steam curing is available in the form of perforated pipes extending longitudinally under each bed, and when this process is in use the anchors can be removed 24 hours after casting. The wires between the sleepers are cut by means of an oxy-acetylene burner. The sleepers in the moulds are then mechani-

the sleepers are taken from the moulds is a 50-tons testing machine, and at least one sleeper from each production-line is tested in accordance with B.S. No. 986.

Ten cranes varying between 5 tons and 10 tons capacity traverse the length of the shop (*Fig.* 11), two 3-tons overhead cranes deliver concrete from the batch mixers across the shop, and a number of 2-tons mobile cranes are in use.

Mixing Plant.

There are two concrete-mixing plants, one on either side of the factory, each comprising high-level storage bins for cement, sand, and two sizes of aggregate. The cement is supplied from an adjacent

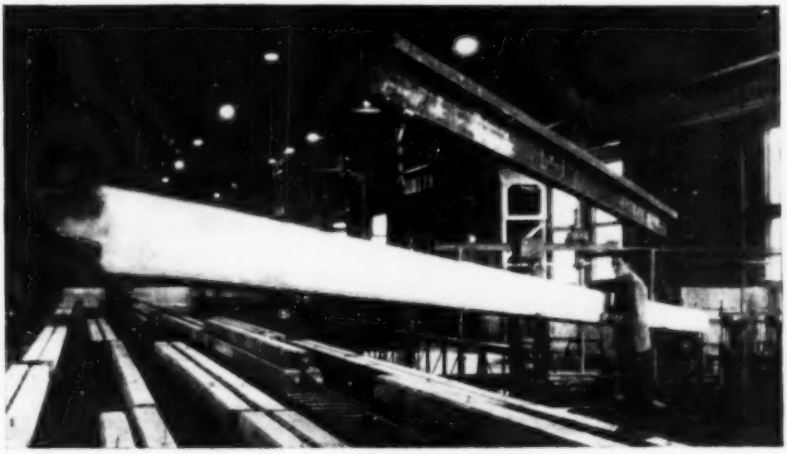


Fig. 11.—Removing 65-ft. Pile from Stressing Bed.

cement factory and the aggregate from nearby whinstone quarries; the sand is from Doune, near Stirling. Cement is delivered loose in tipping vehicles of the type seen to the right of *Fig.* 10. The raw materials are hoisted from bins at ground level by elevators and belt conveyors. The supply of cement from the high-level storage to the batch-weighing machine is controlled by an electrically-operated Archimedian screw with a fine adjustment. The concrete mixers consist of four 14/10 open-pan mixers and two 10/7 mobile open-pan mixers. The concrete is discharged by elevated skips into large hoppers on the 3-tons overhead cranes. The mixture in general use is 1 : 4½ by weight with a water-cement ratio of 0·39 to 0·41. The average crushing strength of 6-in. cubes at five days is 5500 lb. per square inch. With steam curing a strength of 5000 lb. per square inch is obtained in 24 hours. The strength at 28 days averages 9000 lb. per square inch, and a minimum of 4000 lb. per square inch is required before the anchors are removed from the wires. Testing apparatus is available for beams up to 60 ft. long.

The factory is situated between Glasgow and Edinburgh in the Scottish industrial area and has excellent rail and road facilities, while sea transport is available at both east and west coast ports.

Fire Resistance of Concrete Slabs.

THE results of fire-endurance tests on reinforced concrete slabs are given in a report entitled "Fire Resistance of Concrete Floors" recently issued by the U.S.A. National Bureau of Standards. Eleven of the slabs were 8 ft. 8 in. by 4 ft. 6 in., and four were about 17 ft. by 13 ft.; the thicknesses were from 2½ in. to 8 in. All the slabs were cast in situ and were made with gravel aggregates; six of the slabs were supported by lightweight precast concrete beams. The effects of moisture content, the type of aggregate, and plaster ceilings applied directly to the soffits of the slabs are discussed. The larger slabs were tested while carrying loads between 40 lb. and 80 lb. per square foot.

The Price of Cement.

OWING to increases in costs in the cement industry the prices of ordinary and rapid-hardening Portland cement were increased by 2*s.* per ton on May 20, 1953.

A considerable quantity of cement and clinker is being imported at the request of the Minister of Works in order that supplies for the home and colonial markets are available to meet this year's building programme. The imported cement will be sold at the same price as that produced in this country although its cost is considerably higher.

BOOK REVIEWS.

"Rigid Frame Formulas." By A. Kleinlogel. (London: Crosby Lockwood & Son, Ltd. 1952. Price 75s.)

THIS is a translation from the German language of the eleventh edition of Professor Kleinlogel's well-known book, "Rahmenformeln," which was reviewed in this journal for June, 1949. It comprises the data required for the analysis of 114 different types of single-bay rigid frames with fixed or hinged supports and loaded with uniform, triangular, or concentrated loads or for a uniform increase in temperature. Diagrams showing the approximate distribution of the bending moments and directions of the forces are given for each load considered on each type of frame. Users of the data must, of course, have sufficient experience to realise the limitations of such data and use them with discretion.

"Erlauterungen zu den Stahlbetonbestimmungen mit Beispielen." By W. Gehler in collaboration with Christian Palen. (Berlin: Wilhelm Ernst & Sohn. 1952. Price 18 D.M.)

THE sixth edition of this book gives a detailed commentary, illustrated by numerical examples, on the German regulations for reinforced concrete. Whilst many of the conclusions reached are of general application, there are some which cannot be readily adapted to British practice on account of differences in codes of practice and regulations, and in some instances due to different methods of approach to certain problems. The design of prestressed concrete is not included, because no rules appear to have been agreed upon.

The authors point out that phenomena such as shrinkage, creep, and plasticity of concrete have not received much attention in Germany on the ground that extensive research appears to be less necessary in a country where the climate is alleged to be temperate. So far, no serious damage has been recorded which could be attributed to any of these causes.

"Bandesanstalt für Strassenbau Wissenschaftliche Berichte. Heft 1. Betongefüge und Rissebildung in Betonfahrbahndecken." By Rudolf Dittrich. (Berlin. Wilhelm Ernst & Sohn. 1952. Price 4·8 D.M.)

THIS report gives the results of tests carried out on cores cut from road slabs. It does not, however, add to our knowledge of the properties of concrete nor give any new information on the construction of concrete roads.

"Proceedings of the International Congress of Prestressed Concrete, Ghent, 1951." (Obtainable from Concrete Publications Ltd., London. Price 75s.; 15 dollars in Canada and U.S.A.)

AT the international congress on prestressed concrete, held in Ghent in September, 1951, under the presidency of M. Freyssinet, fifty-six papers were presented by engineers from thirteen countries. The papers and the discussions arising from them, with the introductory speeches by Professor Magnel and M. Freyssinet, have been published in a volume of 706 pages, and form a valuable record of the progress made in the theory and in the applications of prestressed concrete throughout the world.

The book is divided into two sections, the first containing papers dealing with the applications of prestressed concrete and the second dealing with design and testing. One of the most interesting papers is that by M. Freyssinet in which a new theory is put forward on the setting and hardening of cement and concrete (this paper was referred to in this journal for April, 1952). Among the novel applications described are an aeroplane runway comprising interconnected prestressed precast beams; an apparatus for determining directly the force in wires during tensioning in which the wires are attached to a ring-dynamometer which in turn is attached to the jack (it is stated that by this method the force in the wires may be determined with an accuracy of 0·4 per cent.); an anchor, developed in Italy, suitable for cables comprising any number and arrangement of wires; and a precast pipe also developed in Italy.

The section dealing with design includes papers on methods of calculating the factor of safety against buckling of box-shaped beams, and the stresses at the ends of prestressed beams. The results are also given of tests of beams under constant and dynamic loads, and of the influence of high temperatures on the strength and deformation of beams.

Nineteen of the papers are in the English language, thirty-two in French, and five in Flemish; summaries of each paper are given in these three languages.

The "Traverse" Method of Stress Analysis.

We have received from Mr. Ralph W. Stewart (formerly Principal Structural Engineer of the City of Los Angeles, U.S.A.) the following account of the method of stress analysis devised by him and which is known as the traverse method.

By utilising the geometry of deformed structures in connection with the laws of bending, some continuous frames can be analysed more rapidly and satisfactorily than by the methods now in general use. Let *Fig.* 1 represent a cantilever beam considered to be bent only by the load P at the free end. The bending-moment diagram is a triangle. If the cross section of the beam is constant the uniform increase in bending moment will cause uniform increase in the rate of curvature of the elastic curve. This gives the curve the properties of the "railway transition curve" which is used to connect the tangents of a railway curve to the arc. A property of this curve is that for small values of the angle of intersection of its tangents (the Δ-angle), its long tangent is twice the length of its short tangent. It is assumed that without measurable error the sum of the lengths of the two tangents to the elastic curve equal the undeflected length of the beam, and that the horizontal displacement of the end of the beam under P is so small that it can be neglected. The Δ-angle is then at the one-third point of the beam nearer to the support. This is a corollary of the conjugate beam theorem in which a beam is subjected to a load the intensity of which, at any point, is equal to the ordinates of the $\frac{M}{EI}$ diagram, the reactions being equal to the end slopes, and the intersections of which will consequently be under the centre of gravity of the $\frac{M}{EI}$ diagram. This also applies to members having a variable moment of inertia.

At any point along the beam the rate of curvature of the elastic curve varies directly with the bending moment and inversely with E and I. The expression $\frac{M}{EI}$ denotes the rate of curvature. The total change in direction of the elastic curve between any two points is equal to the area of the $\frac{M}{EI}$ diagram between such points. Thus the Δ-angle in *Fig.* 1 equals $\frac{Ml}{2EI}$ in which M is the moment at the support and l is the length of the beam. The deflection d is $\Delta\left(\frac{2}{3}l\right) = \frac{Ml}{3E_{\text{I}}}$. This arithmetical procedure is the same as that used in the moment-area method, but the geometrical illustration of bending has been added and the deflection is treated as the product of an angle and a distance.

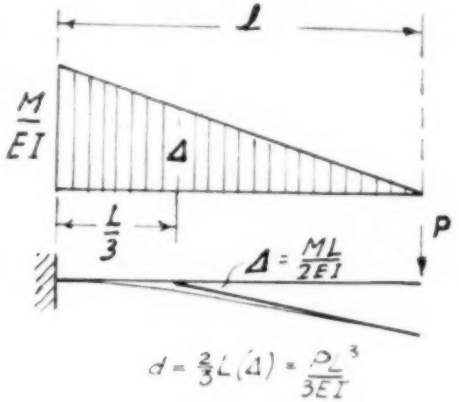


Fig. 1.

A series of straight lines which at certain

critical points are tangential to the elastic curves of a continuous structure is known as a traverse of its elastic curves. The traverse method combines the geometrical properties of this traverse with the laws of bending.

To obtain values of the bending moments the properties of *Figs.* 2(a) to (e) must be investigated. *Fig.* 2(a) represents the traverse of the elastic curve of a beam LR which has constant cross section and is hinged at L. A moment M applied at R causes the beam to bend as shown. The angle Δ is at one-third of the length of the beam from R and is equal to the area of the $\frac{M}{EI}$ diagram which is $\frac{Ml}{2EI}$. In any triangle the sides are proportional to the sines of the opposite angles and in all triangles formed by an elastic curve traverse the angles are so small that their sines, tangents, and arcs can be considered equal; also the cosine of any traverse angle can be assumed to be unity. This means that the lengths of all lines of a traverse are measured as the lengths of their projections on the undeflected axis of the member. Therefore in *Fig.* 2(a) the angles of the end slopes are respectively $\frac{\Delta}{3}$ and $\frac{2\Delta}{3}$.

In the traverse method the stiffness of a member is defined as the ratio of the moment applied to one end of a freely supported beam to the change in direction of the elastic curve which it causes. The change in the direction of the elastic curve is equal to the angle between the tangents at the ends of the curve and is shown as Δ in *Fig.* 2(a). Therefore the stiffness $= \frac{M}{\Delta} = \frac{M}{\frac{Ml}{2EI}} = \frac{2EI}{l}$; also moment ÷ stiffness = Δ, and Δ × stiffness = moment.

In *Fig.* 2(b) an additional moment is applied at the left-hand end of the beam, and the elastic curves and angles Δ due to each moment are shown separately. If these two curves and their traverses are algebraically combined by subtracting the ordinates below the undeflected axis of the beam from the ordinates above it, *Fig.* 2(c) is obtained. This does not change the magnitude of the angles.

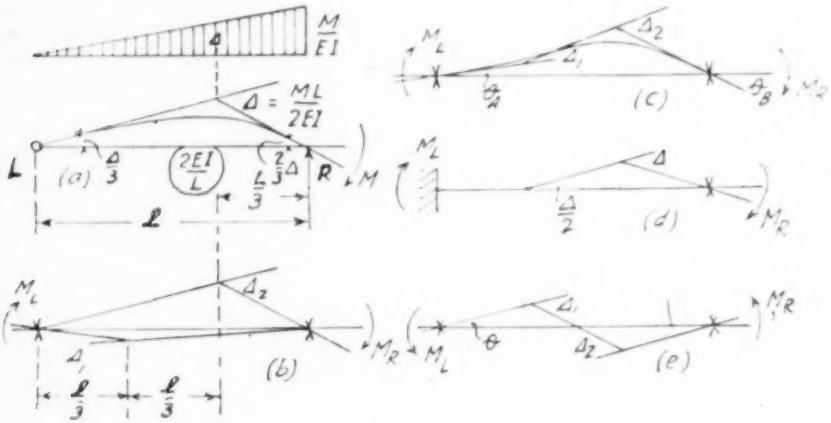


Fig. 2.

Fig. 2(c) represents the most common form of the traverse. If the span has constant cross section the angles Δ are at the one-third points of the beam and from this we have, by geometry, $3\theta_A + 2\Delta_1 = \Delta_2$ and $2\theta_A + \Delta_1 = \theta_2$. If any two angles of this traverse are known the other two may be obtained from the properties of the figure. Therefore if the moment and the rotation of the joint at the left-hand end are known the following may be derived:

$M_L \div$ stiffness $= \Delta_1$.

$$3\theta_A + 2\Delta_1 = \Delta_2 \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$\Delta_2 - \Delta_1 - \theta_A = \theta_B;$$

$$\text{or } 2\theta_A + \Delta_1 = \theta_B \quad . \quad (2)$$

$$M_R = \Delta_2 \times \text{stiffness.}$$

If the left-hand end is fixed the traverse will take the form of *Fig.* 2(d) in which the traverse is an equilateral triangle and $\Delta_1 = \frac{1}{2}\Delta_2$; therefore $M_L = \frac{1}{2}M_R$. If both end moments are clockwise and the smaller is more than half of the larger the traverse takes the form of *Fig.* 2(e). If these moments are equal *Fig.* 2(e) is symmetrical and $\theta = \frac{\Delta}{3}$. A property of the traverse which applies to beams with variable or constant section is that the middle line of the traverse (produced if necessary) intersects the axis of the beam at the point of contraflexure of the elastic curve.

Example.

Fig. 3 shows the application of the traverse method of analysis to a continuous frame. The numerals in small circles near the centres of the members represent the relative values of their stiffnesses. The true stiffness of column AB is $\frac{2E_1I_1}{l_1}$, the subscripts meaning that the values of E, I, and l are those for the first member (AB) used in the analysis. The relative stiffness of each member is obtained by dividing its actual stiffness by $\frac{E_1I_1}{l_1}$; thus the relative stiffness of AB is 2.

Let a load P be applied at the end of the cantilever to the right of E. It is desired to compute the moments on the frame due to this load.

Draw through E a line representing the slope of the elastic curve at E due to the bending caused by the load P. If the beam were hinged at D the traverse of beam DE would be as *Fig.* 2(a), but if beam CD had infinite stiffness so that DE would be fixed at D the traverse would be as *Fig.* 2(d). The actual condition is that beam CD only partially restrains beam DE, so that the traverse takes the form of *Fig.* 2(c). In a similar manner the traverse of each beam and column is sketched proceeding to the left. Assume a rotation of unity at B. By geometry the values of the Δ-angles for column AB are written as shown. Compute the moments at the top and bottom of column AB by multiplying the Δ-angles by the stiffness (2) and record the values at the ends of the member. The moments about B are then balanced, a moment equal to 4 units being required at the end B of the beam BC. The Δ-angle at the left of beam BC is obtained by dividing the moment 4 by the stiffness 4 = 1. The Δ-angle at the right of the beam is found by equation (1) to be 5. From equation (2) the rotation of joint C is 3. The Δ-angles in columns CF and CG follow from geometry, and the end moments on the columns are found by multiplying the Δ-angles by the stiffnesses.

The signs of the moments and the value of the balancing moment about C are determined by considering the bending which would follow the removal of the load P. If this load were removed all the beams would return to their undeflected form and the apex of each triangle would move about C in the direction shown by the arrows. For members CF, CG, and CB the direction of these arrows indicates counterclockwise rotation about C, but the arrow on CD indicates clockwise rotation about C. The moment M_{CD} must therefore be opposite in direction to the other three moments and, for equilibrium, must be equal to their sum, that is 35. Having this moment and the joint rotation at C the bending of CD can be evaluated and the procedure continued to E where the moment is found to be 347.

The actual moment at E is Pl_2 and by multiplying all moments on *Fig.* 3(a) by $\frac{Pl_2}{347}$ the moments throughout the frame due to P are obtained. The deflection under P can now be computed. The relative value of the slope of the elastic curve at E is $64\frac{1}{3}$, as shown. Its actual value is obtained by dividing the relative value by $\frac{E_1I_1}{l_1}$, which is the factor omitted from the stiffness value of column AB. Multiplying the actual value of the slope by the length l_2 of the cantilever and adding the deflection due to the curvature of the cantilever itself gives

$$\frac{Pl_2}{347}\left(64\tfrac{1}{3}\,\frac{l_1I_2}{E_1I_1}\right) + \frac{Pl_2^3}{3E_2I_2}$$

for the deflection under P.

In *Fig.* 3(b) the Δ-angles on each side of B indicate counterclockwise rotation about this joint. The sum of their values

is a measure of the resistance to bending of the members meeting at the joint, and the moment they represent must be balanced by an externally applied moment of equal numerical value but opposite sign. The sum of these moments is − 347 and is balanced by an external moment of + 347, which is the same as the moment at E in *Fig.* 3(a). This is in accordance with Maxwell's theorem of reciprocal deflections, and this check is a feature of the traverse method.

The externally applied moment of 347 at B is shown as if it acted through an arm projecting from B. In a structure it would usually be due to a load on the

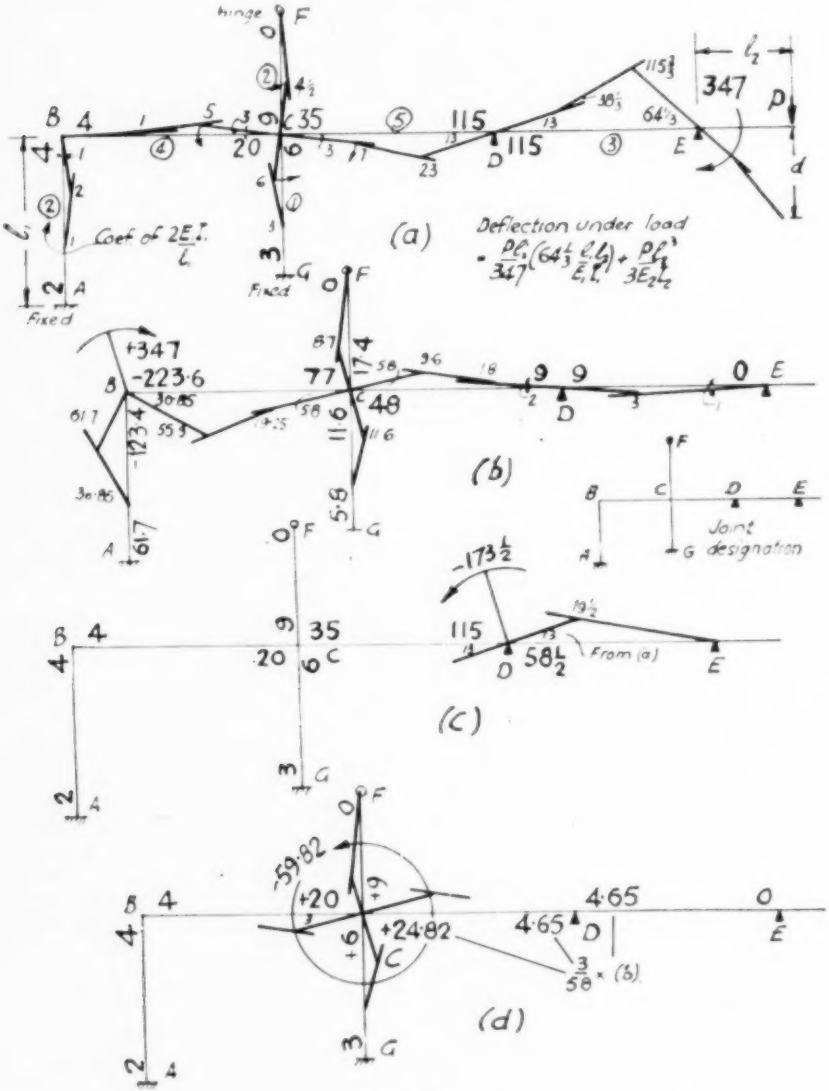


Fig. 3.

beam BC or the column AB, and would be the fixed-end moment at B due to the load on a member at joint B. If the externally applied moment of + 347 were the fixed-end moment at B due to loads on BC, this moment would be resisted at the joint by moments of − 223·6 on BC and − 123·4 on AB. Thus the sum of the moments about B would be zero. After *Figs.* 3(a) and (b) are completed the calculations are nearly completed. For *Fig.* 3(c) the moments from A to D and the rotation of the joint at D are copied from *Fig.* 3(a). The traverse for span DE is drawn for the rotation of 13 at the joint and the two clockwise moments, represented by the Δ-angles, are added together resulting in the externally applied moment of − 173·5. This is half of the moment of 347 at B in *Fig.* 3(a), as it must be by Maxwell's theorem.

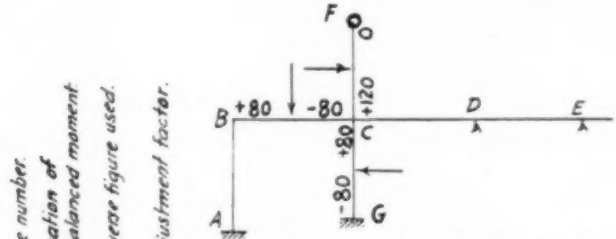
For *Fig.* 3(d) the moments from A to C (except the moment of 35 in span CD) and the rotation of 3 at joint C are copied from *Fig.* 3(a). The traverse from E to C in *Fig.* 3(b) yields a rotation of 5·8 at C. This traverse is made to agree with *Fig.* 3(d), which has a rotation 3 at C, by multiplying the values of the moments on spans CD and DE in *Fig.* 3(b) by $\frac{3}{5 \cdot 8}$. The moments measured by the Δ-angles at C are then added together to get the external balancing moment − 59·82.

Applications to Numerical Problems.

In *Fig.* 4 each of the members BC, CF, and CG of the frame previously considered is assumed to have a length of 16 units. Each is loaded at its centre with a concentrated load of 40 units, the direction of the loads being shown on the figure. (The units of weight and length are not specified as the analysis applies to any system of weights and measures. The lengths of the members which are not loaded need not be specified as their stiffnesses, given in the general analysis before, include all the necessary data for analysis.) For members BC and CG, which are restrained at both ends, fixed-end moments of 80 units are used as the applied external loads. For member CF the moment at C, when C is fixed and F is free, is 120 units.

The sum of the applied moments about C is − 80 + 120 + 80 = + 120 units. This is taken as the "unbalanced" moment applied at C.

Line 1 in the table shows the moments throughout the frame caused by the fixed-end moment of 80 units at B. Line 2

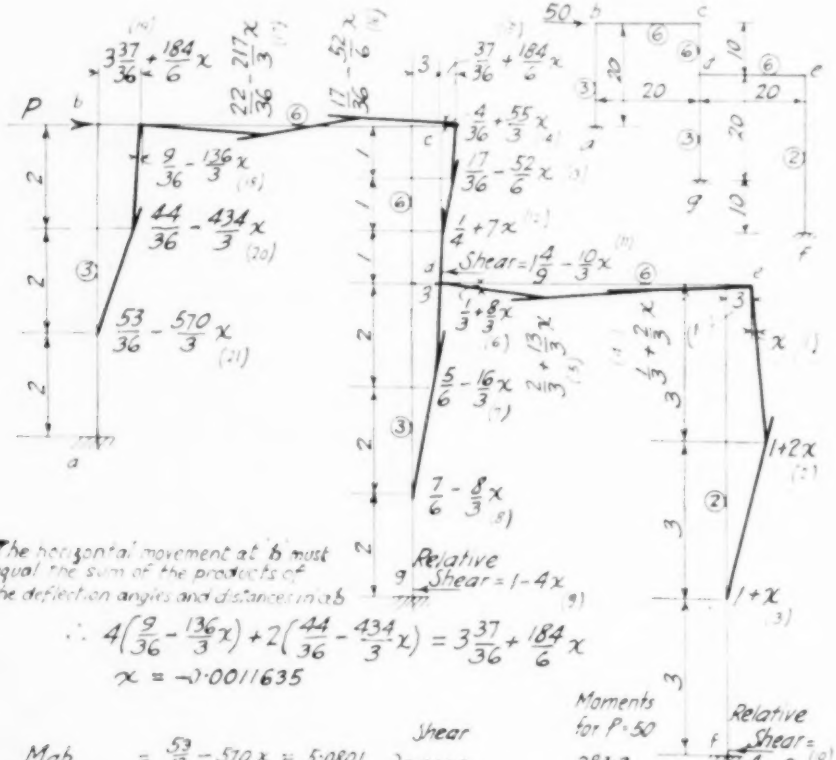


Line number.	Location of unbalanced moment.	Traverse figure used.	Adjustment factor.	M_{BC}	M_{CB}	M_{CF}	M_{CG}	M_{CD}	M_G	M_{DC}
1	B	3(b)	$+\frac{80}{347}$	-51.5	-17.8	+ 4.0	+26	+11.1	+ 13	+ 7.1
2	C	3(d)	$-\frac{120}{59.82}$	-8.0	-40.1	-18.0	-120	-49.8	- 6.0	- 9.3
3				+80.0	-80.0	+120.0	+80.0	0	-80.0	0
4				+20.6	-137.9	+106.0	+70.6	-38.7	-84.6	+8.8

Fig. 4.

does likewise for the unbalanced moment of 120 units at C. Line 3 records the fixed end moments shown on the figure. Line 4 is the sum of the moments in lines 1, 2, and 3, and gives the final moments due to the loads. As a check it is noted that the moments about joint C are balanced.

Fig. 5 shows a solution of a terraced frame. The number of unknowns which would have to be evaluated by the common methods is six (four joint rotations and two deflections). The traverse solution uses one unknown and a final adjustment—the equivalent of two unknowns—and there are no simultaneous equations to be solved. The calculation commences at joint e with an arbitrary displacement of 3 units and a rotation of x units. The solution follows the lines of the previous problem and the order in which the calculations are made is indicated by the numbers in brackets against the expressions written on the


The horizontal movement at b must equal the sum of the products of the deflection angles and distances in ab
Relative Shear = 1 - 4x
Shear = 1⅘ - 10/3 x
Relative Shear =
Moments for P = 50
Shear
Check
Mab = 53/12 - 5·10x = 5·0801
Mba = Mbc = 44/12 - 4·34x = 4·1718
Mcb = Mcd = 17/6 - 5·2x = 2·8939
Mdc = 9/6 + 4·2x = 1·4510
Mde = 4 + 2·6x = 3·9697
Mdg = 15/6 - 1·6x = 2·5186
Mfg = 21/6 - 8x = 3·5093
Med = Mef = 2 + 4x = 1·9953
Mf = 2 + 2x = 1·9977
·4626
·4345
·8971 = P
·3014
·1331
·4345
283·3
233·0
161·3
80·9
221·2
140·4
195·6
111·3
111·4

$$\therefore\ 4\left(\frac{9}{36}-\frac{136}{3}x\right)+2\left(\frac{44}{36}-\frac{434}{3}x\right)=3\frac{37}{36}+\frac{184}{6}x$$

$$x=-0{\cdot}0011635$$

Fig. 5.—A Terraced Frame.

figure. This problem has been published elsewhere with a solution by moment distribution.*

Application to Derivation of Formulæ.

The use of the traverse method is not restricted to the solution of numerical problems. It can be used to derive formulæ. In *Fig.* 6(a) the stiffnesses of the members, the story heights, and the loads are represented by symbols. The formulæ for the moments at the ends of the members are derived on the figure. By substituting numerical values for the stiffnesses and story heights in the formulæ for the moment at the base of the lower column, the formula can be solved for x (which represents the rotation of the joints at the top corners) and all the bending moments may be evaluated,

* "Automatic Design of Frames." By L. E. Grinter (p. 26).

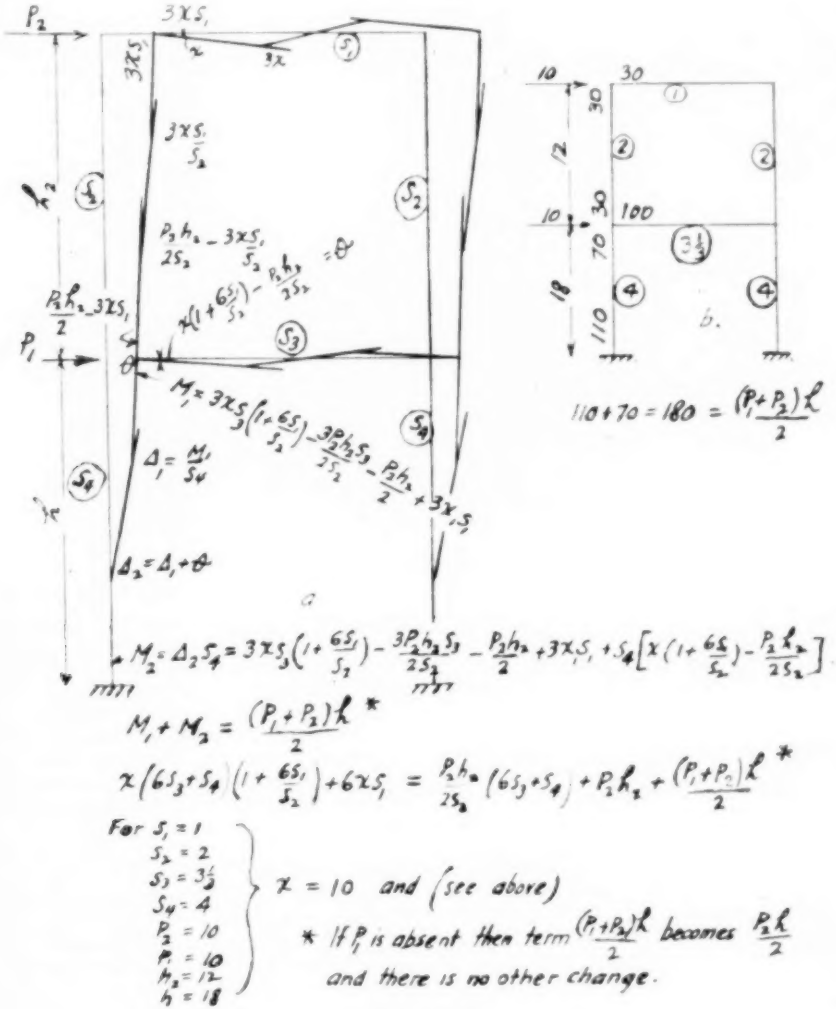

a
b.
$M_1 = 3xS_3\left(1+\frac{6S_1}{S_2}\right) - \frac{3P_2h_2S_3}{2S_2} - \frac{P_2h_2}{2} + 3xS_1$
$\Delta_1 = \frac{M_1}{S_4}$
$\Delta_2 = \Delta_1 + \theta$
$M_2 = \Delta_2 S_4 = 3xS_3\left(1+\frac{6S_1}{S_2}\right) - \frac{3P_2h_2S_3}{2S_2} - \frac{P_2h_2}{2} + 3xS_1 + S_4\left[x\left(1+\frac{6S_1}{S_2}\right) - \frac{P_2h_2}{2S_2}\right]$
$M_1 + M_2 = \frac{(P_1+P_2)h}{2}$ *
$x\left(6S_3+S_4\right)\left(1+\frac{6S_1}{S_2}\right) + 6xS_1 = \frac{P_2h_2}{2S_2}\left(6S_3+S_4\right) + P_2h_2 + \frac{(P_1+P_2)h}{2}$ *
For $S_1 = 1$, $S_2 = 2$, $S_3 = 3\frac{1}{3}$, $S_4 = 4$, $P_2 = 10$, $P_1 = 10$, $h_2 = 12$, $h = 18$
$x = 10$ and (see above)
* If P_1 is absent then term $\frac{(P_1+P_2)h}{2}$ becomes $\frac{P_2h}{2}$ and there is no other change.

Fig. 6.

The stiffnesses, loads, and dimensions and the resulting moments are shown in *Fig.* 6(b). With the Δ-angles evaluated the deflections are quickly obtainable.

For structures comprising members of variable moment of inertia the Δ-angles will not be at the one-third points of the spans of the members, and an unsymmetrical member will have two stiffnesses, namely one at each end.

[More detailed accounts of this method are given by the author in "The Analysis of Continuous Structures by Traversing their Elastic Curves," published by the author at 1200 Arapahoe Street, Los Angeles 6, Calif., U.S.A. (price 5 dollars), and in a German translation entitled "Die Traversen-Methode," published by Wilhelm Ernst & Sohn, Berlin, price 17.50D.M.]

A Long-Span Hollow-Beam Roof.

THE roof of a gymnasium at Chiswick Polytechnic, London, shown in *Fig.* 1, has a clear span of 40 ft. 9 in. and is built of hollow prestressed beams 42 ft. 4 in. long. These are believed to be the longest hollow beams yet made. The beams are 14 in. wide and have a depth of 8½ in. at the ends increasing to 10½ in. at the middle. The cavity is 9½ in. wide and varies in depth from 5½ in. at the ends to 6½ in. at the middle. Each beam is prestressed by fourteen high-tensile wires of 0·2 in. diameter and is designed to carry an imposed load of 25 lb. per square foot plus a dead load due to the weight of the beam and the finishes of 68 lb. and 28 lb. per square foot respectively. The beams were tested with a load equal to the dead load plus twice the imposed load. No cracks were then visible. The deflection due to the dead loads and an imposed load of 25 lb. per square foot was 0·435 in., that is less than $\frac{1}{1100}$ of the span.

A mobile crane was used for placing the beams in position and all the beams for the roof, which has an area of 330 sq. yd., were placed in less than a week, after which the joints between the beams were grouted. The architects for the building are Messrs. Dalgleish & Pullen, F.F.R.I.B.A., in collaboration with Mr. C. G. Stillman, F.R.I.B.A., Architect of the Middlesex County Council. The beams were made by Concrete, Ltd., by the Company's patented processes, and the builders are Messrs. F. Troy & Co., Ltd.

Fig. 1.—Placing 42-ft. Beams on a Roof.

Proposed Reservoir in British East Africa.

APPLICATIONS are invited for permission to tender for the construction of a reservoir of 12,000,000 gallons capacity at Mombasa, British East Africa. Alternative tenders are required for (a) the construction of a reinforced concrete reservoir, and (b) the design and construction of a prestressed concrete reservoir. Further information may be had from the Board of Trade, Commercial Relations and Exports Department, Horse Guards Avenue, London, S.W.1; the reference number CRE/36940/52 should be quoted.

The "Vacuum Concrete" Processes.

In the "vacuum" processes for treating concrete, devised by Mr. K. P. Billner, of Philadelphia, water in excess of that required for the hydration of the cement is extracted from the concrete immediately after it is placed. This is done generally by lining one or more faces of the shutter with a filter, which comprises wire mesh and a gauze fabric, connected to a vacuum pump as shown diagrammatically in *Fig.* 3. As the pump is operated the concrete is compacted by atmospheric pressure, which in some cases may be assisted by the weight of the concrete. In untreated fresh concrete the atmospheric pressure is resisted by the intergranular pressure between the pieces of aggregate and the hydrostatic interstitial pressure in the minute channels in the concrete. The action of the vacuum is to reduce the interstitial pressure and so compel the concrete to contract within the mould, thus expelling part of the water through the filter. This effect penetrates the mass of the concrete but at a diminishing speed similar in manner to the progressive increase in temperature along the length of a metal rod heated at one end.

The expulsion of the water and the contraction of the concrete result in the production of a compact mass with a high cohesive strength allowing the immediate removal of shutters of elements up to 18 ft. high, and also reduces the water-cement ratio and so improves the properties of the hardened concrete. The compressive strength of vacuum-treated concrete at an age of eight or nine days is, for example, equal to that of untreated concrete with the same cement content at 28 days, and the final strength is usually increased by 1200 lb. to 1500 lb. per square inch.

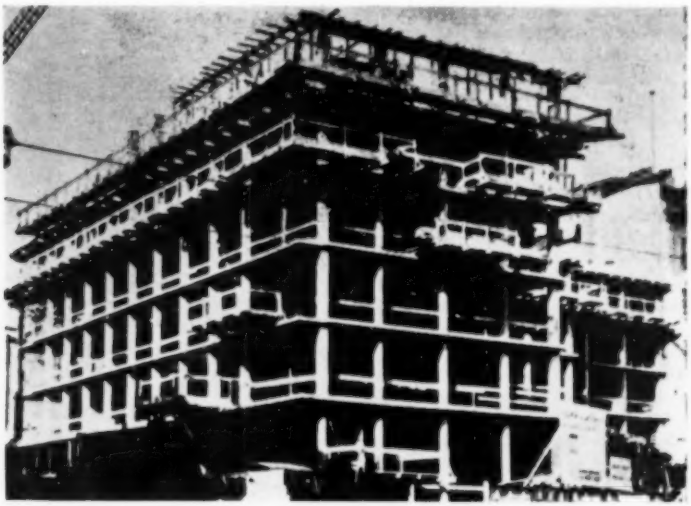


Fig. 1.—Office Building at Geneva.

In the following are given descriptions of some recent applications of the process.

Framed Building in Switzerland.

Floors.—This is part of a nine-story building at Geneva (*Fig.* 1), known as the "Mont-Blanc Centre", for use as offices. The frame and the floors are of reinforced concrete and the vacuum processes were used in their construction. The floors are 5½ in. thick. The concrete was of a rather wet mixture cast in wooden shutters, compacted by immersion vibrators, and screeded. The mats

for the surface treatment were then placed in position. These mats were of wood with an air-tight lining of hardboard on the lower face, and this lining was fitted with the filter connected to the vacuum pump. Each mat was connected by a flexible armoured-rubber hose to a distributing manifold in the form of a sheet-steel pipe with nozzles for supplying the hoses. The mats were placed side by side and moved forward (*Fig.* 2) in leap-frog fashion after having been in operation for about fifteen minutes on the fresh concrete. As soon as the treatment was finished, a tap controlling the suction in the mat was closed and the mat was lifted and set in its next position; it was possible, at that stage, to walk on the treated concrete and to screed the surface. This screeding, followed by smoothing with a trowel, gives a surface almost equal to a mortar rendering. The surface is less liable to cracking than an applied topping because the floor is monolithic. This treatment enables fairly wet concrete to be used, so economising in labour in placing and screeding, while obtaining a dry and dense concrete of high quality. In this case the centering was struck after seven days, but the concrete was strong enough at four days to permit this to be done. The first column of this building (*Fig.* 1) was cast on July 7, 1952, the photograph was taken on August 12, and the frame, floors, and roof were completed on September 5.

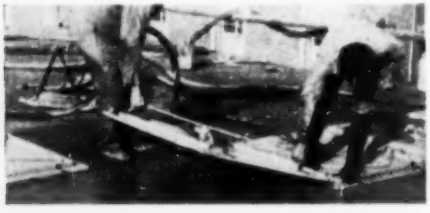


Fig. 2.—Removing Vacuum Mats 15 Minutes after Placing the Concrete.

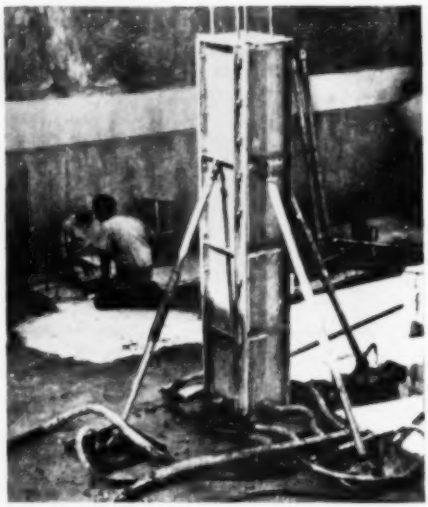


Fig. 4.—Shuttering for a Column.

The largest of the mats had an area of 3 ft. 4 in. by 4 ft. 8 in., and eight mats in all, making a total area of 14½ sq. yd., were used. The concrete in the floors had a cement content of 420 lb. per cubic yard. These floors will be covered with rubber fixed directly to the concrete. The roof slabs had a cement content of 506 lb. per cubic yard.

Columns.—The columns are 8 ft. 10 in. high; 288 have a cross section of 1 ft. 2 in. by 1 ft. 2 in., and 154 have a cross section of 8 in. by 1 ft. 2 in. Whilst casting a floor, a part of each column, ¾ in. high, was formed on the top of the floor, with the splicing bars projecting. The preassembled reinforcement for the column was erected, followed by the shutters (equipped with the vacuum devices) butted against the bases of the columns. The sides of the shutter were wedged together, and the box fastened to the ground by means of three vacuum-holders. Each vacuum-holder was attached to the

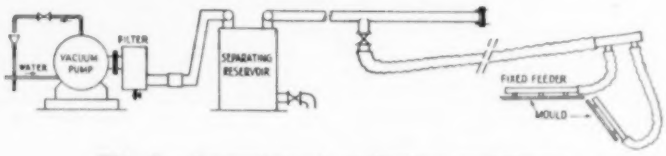


Fig. 3.—Arrangement of Shutter and Pump.

lower end of an adjustable stay (*Fig.* 4). The vacuum-holders secure the shutters to the floor, and adjustment of the length of the stays enables the column to be plumbed quickly. The shutters were made of wood lined with hardboard and stiffened by steel angles.

The shutters were filled at the top through a hopper, and the concrete, which contained 506 lb. of cement per cubic yard, was compacted by a poker-vibrator. When the shutter was full, the suction was brought into action on the four faces and the concrete was further compacted by the effect of the atmospheric pressure whilst at the same time the excess water was expelled. After 15 to 17 minutes of this treatment the concrete was dense enough to permit the shutters to be removed. Because the shutters are removed while the concrete is fresh they come away easily and they may be used many times. Also the concrete, although dense and self-supporting, is plastic, and surface flaws can easily be made good. Seventeen columns were concreted each day of 8 hours with the use of four sets of shutters. The columns were loaded three days after concreting. The contractors were Entreprise Ed. Cuenod, S.A.

Houses in Colombia.

A Colombian architect, Sr. M. Ortega, used the processes in the construction of several hundred small houses in Colombia, and a similar scheme is now in progress in Venezuela. The first undertaking in

Fig. 5.—Erecting a Wall Slab.

Fig. 6.—Casting Roof Slabs.

Colombia comprised 100 houses of the same type, each containing four rooms, a kitchen, and a bathroom. The area of each house is about 500 sq. ft., of which 400 sq. ft. are roofed by an arched slab. The total area of the walls is 800 sq. ft. The houses are built entirely of prefabricated parts; the wall slabs are from 21 sq. ft. to 130 sq. ft. in area, and the curved roof slabs are 13 ft. by 10 ft.

The first stage was the construction of a concrete floor for a group of six houses, and on this floor were cast one slab of each type. These were taken from their moulds with a vacuum lifter and placed on the ground nearby to serve as mould-bottoms for casting all the other slabs of the same shape. Metal side forms, 2½ in. high, were fixed around and above each of the slabs to permit the casting on top of a similar unit, and so on until eight to ten slabs were cast on top of one another in one day. A sheet of paper was placed between each slab. The slabs were reinforced with 0·235-in. diameter wires at about 15-in. intervals in both directions. The concrete was a 1 : 3 : 5 mixture with a cement content of about 405 lb. per cubic yard.

The slabs were subjected to vacuum treatment for 12 to 15 minutes by means of 3-ft. by 4-ft. standard mats. The surface was then hand floated; no other finish was specified for the walls. By means of a vacuum lifter (*Fig.* 5) the wall slabs were erected in their permanent positions 24 hours after they had been cast, and the joints concreted.

The curved roof slabs are 1¼ in. thick, reinforced with wires 7 in. to 8 in. apart. The roof slabs are made of richer concrete (506 lb. per cubic yard) than the walls. The method of casting was similar to that of the walls. As many as 30 segments were cast in one stack (*Fig.* 6)

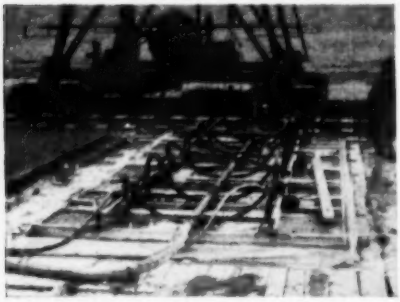


Fig. 7.—Vacuum Treatment of Slab of 27 sq. yd.

Fig. 8.—Transporting Slab of 27 sq. yd.

using only one curved wooden base. The vacuum lifter was made adaptable to small differences of curvature by means of adjustable screws. Door and window openings are formed by vertical shutters.

Lining a Canal in France.

The embanked part of the Ottmarsheim canal (France) is about nine miles long and 118 ft. wide measured on the slope. A semi-flexible revetment was required which would be capable of following small movements and at the same time remain watertight. The work required 620,000 sq. yd. of lightly-reinforced slabs, each having an area of 27 sq. yd. and a thickness of 3½ in. The manufacture and laying of these slabs had to be done in about eighteen months.

Vacuum concrete processes were used, with fixed moulds and mats for the treatment of the fresh concrete, and vacuum lifters for transporting the slabs. The water-cement ratio was reduced by the treatment from 0·54 to 0·40. Two layers of 0·196-in. diameter wires forming a mesh 8 in. by 4 in. were inserted to resist lifting stresses.

The procedure of manufacture was as follows: (1) Fabricating the reinforcement; (2) Placing the side forms, the reinforcement, and the inter-slab connecting bars; (3) Placing and screeding the concrete; (4) Treatment by the vacuum mat (*Fig.* 7); (5) Removal from mould by the vacuum lifter (*Fig.* 9) and placing on lorry; (6) Haulage to store and stacking; (7) Removal from stock, loading on trailer, haulage to site, and setting on the bank (*Fig.* 8).

The average rate of production was one slab, 27 sq. yd. in area and 5 tons in weight, every 4½ minutes. It is estimated that the casting yard was only one-eighth of the area that would have

Fig. 9.—Removing Slab from Mould.

been required if the vacuum process had not been used, and that 30 per cent. of steel was saved.

A Culvert in France.

The process was used for placing in a culvert a length of 6000 ft. of the Bievre stream, in France. The invert is semi-circular and was constructed of in-situ concrete using travelling wooden shutters 98 ft. long faced with sheet steel (*Fig.* 10). The upper part is of parabolic shape cast on the invert in lengths of 8 ft. One shutter was sufficient for concreting five or six lengths every shift of 8 hours. After treatment by vacuum for about 15 minutes the shutters were removed; a 12-in. gap was left between each section which was subsequently filled by a specially-cast ring. The parabolic mould was composed of two shutters travelling on rails in the trough of the invert for the intrados and on the abutments of the invert for the extrados. Hydraulic jacks were used for adjusting the level of the shutters and for striking them. An 8-h.p. pump produced the required vacuum.

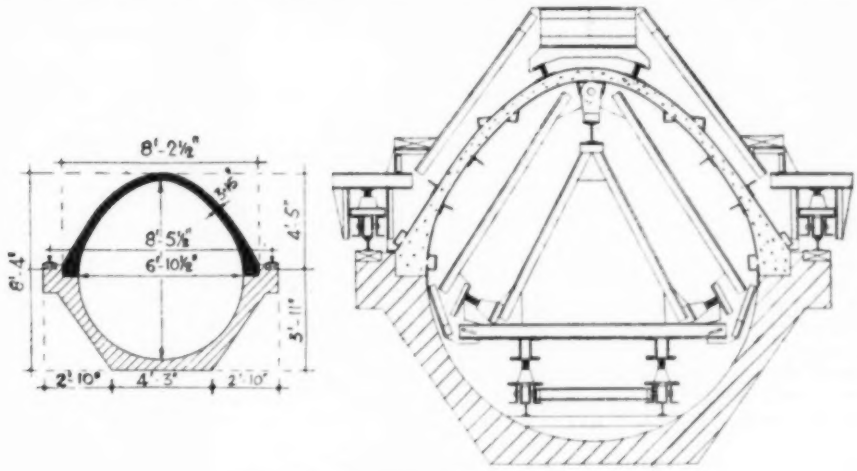


Fig. 10.—Shuttering for a Culvert.

Well Lining in Germany.

A well at Hamburg was sunk through a stratum of peat and lined with concrete of great impermeability in order to prevent the infiltration of humic acids. The well has an internal diameter of 13 ft. and a depth of 82 ft. and was lined with a thickness of 16 in. of concrete treated by vacuum.

The work of excavating and lining comprised the following stages. (1) Casting on the ground of a ring 22 in. high and removing the mould after thirty minutes' vacuum treatment; (2) Erection of the mould six times each day in order to obtain an initial section of lining of a height of 11 ft.; (3) On the next day, the sinking of this ring and the excavation of the material inside the well. These daily operations were repeated until the required depth was obtained. The whole of the lining was cast with one set of plywood shutters 27½ in. high, of which the top 22 in. were equipped, both inside and outside, with the fittings for vacuum treatment. Two hours after removing the mould a hammer and chisel were required to make an impression on the concrete.

Iron-Portland cement, with an air-entraining powder, was used. The concrete was compacted with poker vibrators. Vibration was not applied during the vacuum treatment which was carried out on the two faces and lasted from 30 to 40 minutes. The volume of treated concrete in a ring 22 in. high is 3·9 cu. yd., and from this 16½ gallons of slightly turbid water were extracted. In the

course of the treatment, the level of the concrete fell about 0·4 in. in the mould.

As soon as the vacuum was cut off, the mould was removed and the filter cloths brushed before re-use. These cloths did not appear to be damaged after having been used 40 times. The moulds overlapped the previously-treated concrete by 6 in. and were fixed to vertical guides; under these guides, the concrete which had not been directly treated had a poor surface. Six sections 22 in. high were cast each day of 8 hours.

The "Vacuum Concrete" processes are protected by patents and are used under licence.

Fire Resistance of Steel Columns Encased in Concrete.

THE results of tests to determine the fire resistance of steel columns encased in concrete made with a siliceous aggregate are given in Building Materials and Structures Report No. 124 of the National Bureau of Standards of the U.S.A. Four 6-in. by 6-in. columns of H-section were tested, with a concrete cover to the flanges of 2 in., the concrete being filled solidly in to the re-entrant angles. The aggregates used were river gravel and crushed quartz.

The load of 36½ tons imposed on the columns during the test was equal to the allowable load for the size of column and effective length as calculated by the formula of the American Institute of Steel Construction. The load was applied before starting the fire test and was maintained until rapid yielding of the column occurred. One column with gravel aggregate concrete was loaded to 73½ tons until the temperature of the steel was 833 deg. F., when the load was reduced to 36½ tons.

The fire-endurance limits of the columns with river gravel aggregate was somewhat greater than those with quartz aggregate. For the former yielding occurred at 3 hours 34 minutes and 3 hours 32 minutes and for the latter at 3 hours 10 minutes and 2 hours 50 minutes. The average temperatures in the steel of the first two columns were 1346 deg. F. and 1337 deg. F. respectively and for the second two columns 1157 deg. F. and 1148 deg. F. The maximum temperature recorded in all columns was about 1400 deg. F. Spalling of the concrete did not occur, as had been reported in previous tests with siliceous aggregates, and the cracking was not sufficient to expose the shaft before the critical temperature of the steel was attained. The results indicate that concrete made with siliceous aggregate provides an acceptable material for the protection of structural steel against fire.

Timber Silos for Cement.

IN the journal "Beton-und-Stahlbetonbau" for December 12, 1952, details are given of portable silos for cement which are now being made in Germany. The silos are constructed of timber staves held together by metal bands. The advantages claimed for these silos are that they are easily transportable because of their

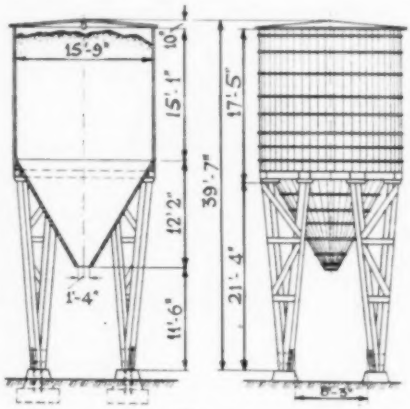
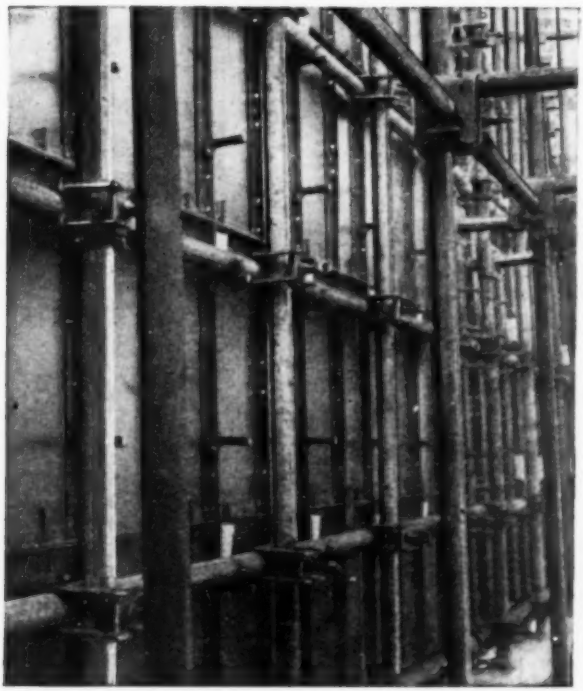


Fig. 1.

light weight, that conveyors may easily be fixed to them, and adaptations simply made. It is also stated that there is less condensation on the inside than is the case with sheet-steel silos; consequently there is less likelihood of a crust of hardened cement forming which would impede the flow of the cement. The silos are supported on timber frames and are made with capacities from 25 tons to 250 tons; the silo shown in *Fig.* 1 has a capacity of 150 tons.

STEEL FORMS

The Multiple System of Interlocking STEEL SHUTTERING for in-situ concrete construction



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A Heavy Building Frame in Prestressed Concrete.

In an article by Berthold Neunert in a recent number of "Beton und Stahlbetonbau" is described the rebuilding of the head office of a bank in Berlin, in which the lower stories are prestressed concrete portal frames. The building (*Fig.* 1) contains a basement, a ground floor, and seven upper floors. It is about 119 ft. high above ground level, and about 150 ft. long. The upper stories are supported by two intermediate rows of columns which are carried by a beam at first-floor level. The beams and columns of the basement and ground floor form portal frames spaced at 21 ft. centres with columns fixed rigidly to the foundation. The ground floor acts as a tie between the columns (*Fig.* 2). Above the first floor level the building is a reinforced concrete frame. The portal, which is four-times indeterminate, comprises reinforced concrete columns and a prestressed concrete beam spanning a

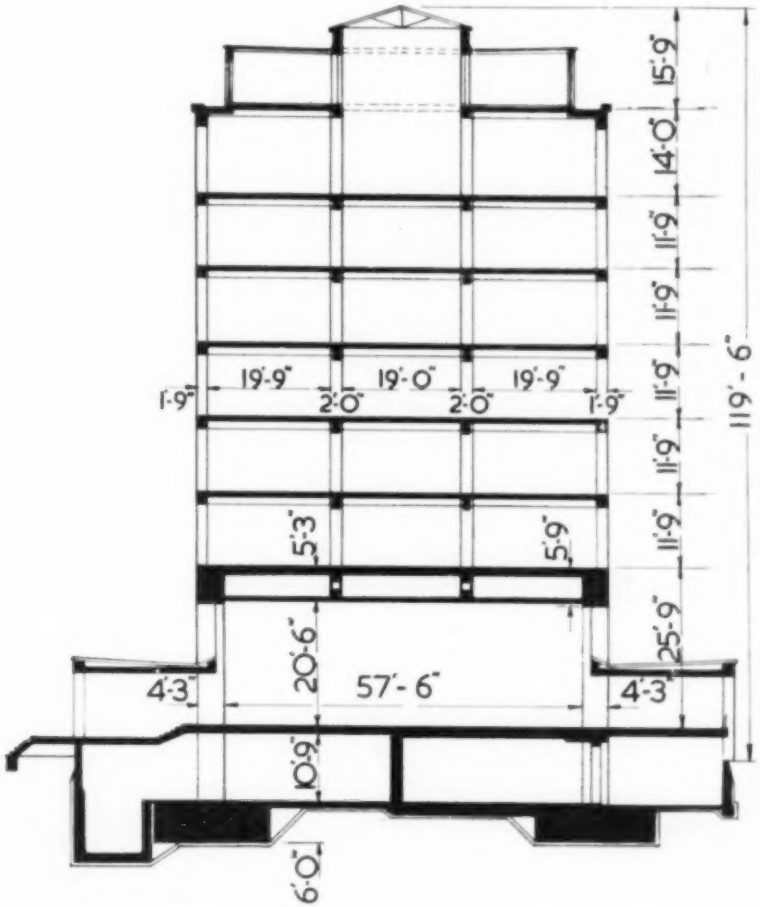


Fig. 1.—Cross Section.

clear distance of 57 ft. 6 in. The load from each of the inner columns supported by the frame is about 350 tons. The prestressed concrete beam is of box section with two webs each 2 ft. 1 in. wide and spaced 5 ft. 3 in. apart (*Fig.* 3). The lower slab between the webs is 6 in. thick. The remainder of the ceiling is of lightweight concrete made with crushed brick aggregate and is not considered to contribute to the strength of the beam. The top slab is 10 in. thick and forms the first floor. The columns of the frame are 4 ft. 3 in. deep by 2 ft. 3 in. wide. They are very slender compared with the beam, and as they have to support heavy vertical loads with only small bending moments it was possible to construct them of reinforced concrete. The intermediate columns are supported by short reinforced concrete cross beams between the webs of the main beam. All the reinforcement in the reinforced concrete is mild steel. The main beams are prestressed by the system devised by Messrs. Dyckerhoff & Widmann, using high-tensile steel known as "90" steel.

The arrangement of the bars in the main beams is shown in *Fig.* 4 and comprises 96 bars arranged in several layers. The lowest layer is straight and the others

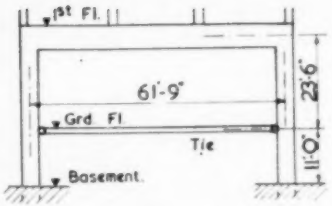


Fig. 2.—Prestressed Frame.

are bent up. The bars are 1 in. diameter with a tensile strength of about 57 tons per square inch and a yield point of about 41·3 tons per square inch. The force in each bar is 20 tons. The anchors are nuts and plates; the threads at the ends of the bars were rolled on in the mill in order not to weaken the bar. The bars are placed in thin metal tubes of about $1\frac{1}{4}$ in. diameter. The method of construction is shown in *Fig.* 5. The whole of the mild steel and the high-tensile steel was placed in the shutters, the lower slab was cast, and the shutters to the sides of the webs were then erected. Lastly the webs and the top slab were cast. After ten days, when the concrete had a strength of 5700 lb. per square inch, the bars were tensioned

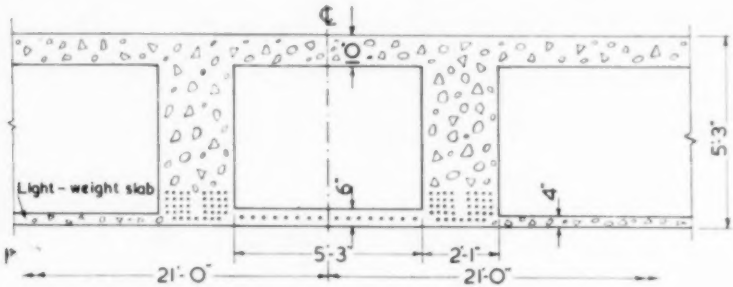


Fig. 3.—Cross Section through Main Beam.

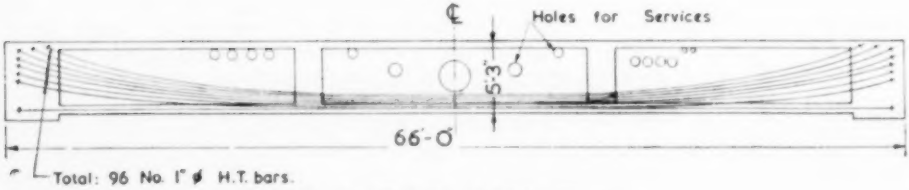
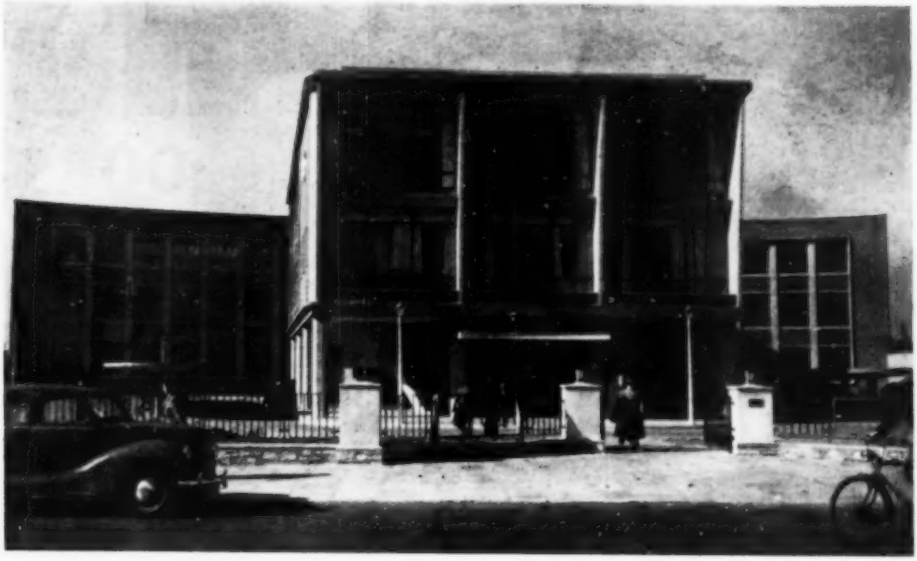


Fig. 4.—Details of Main Beam.



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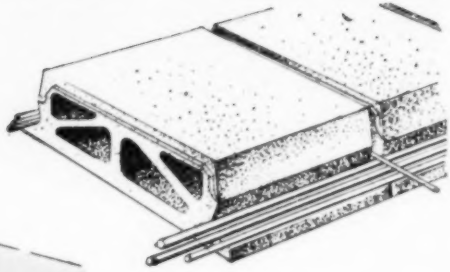

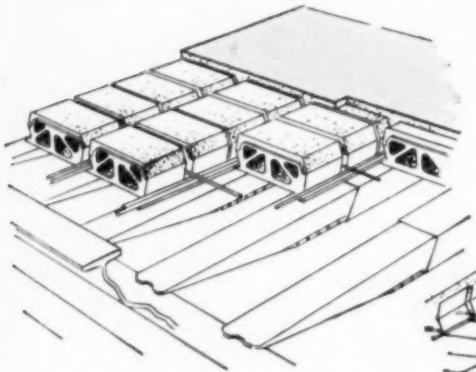

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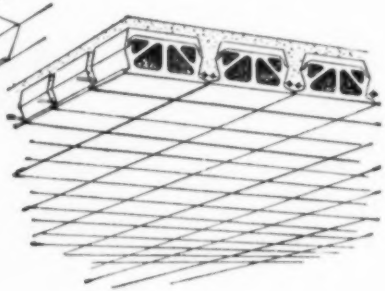

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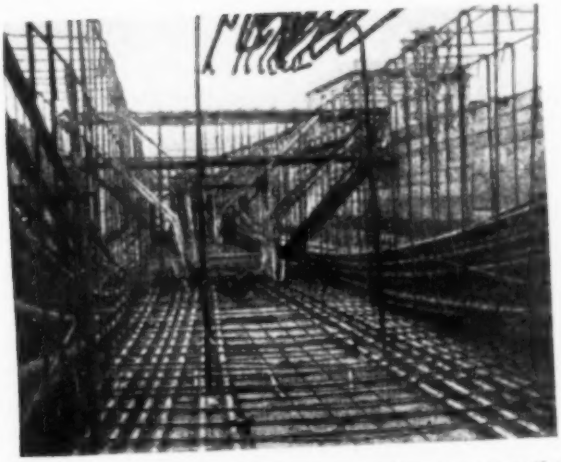


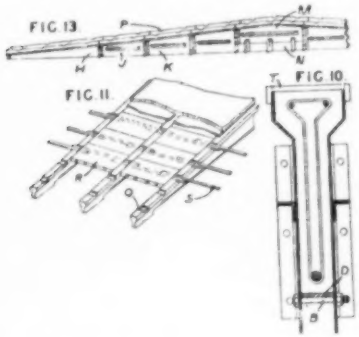
Fig. 5.—One of the Main Beams during Construction.

to one-half of the final tensioning force, and about two months later, after four of the upper floors were completed, the bars were tensioned to the full force. The spaces between the bars and the tubes were then grouted.

The bending moments on the frame are: At the centre of the beam, 4200 tons-ft.; At the junctions of the beam and columns, approximately 1400 tons-ft.; At the feet of the columns, approximately 460 tons-ft. The maximum stresses in the beams due to the dead and live loads, wind, and prestressing, were approximately 1550 lb. per square inch and 183 lb. per square in. compression at the top and bottom respectively, before creep and shrinkage had taken place, and 1580 lb. per square inch compression and 315 lb. per square inch tension after allowing for creep and shrinkage.—Abstracted by Mr. L. J. Elgin, A.M.Inst.C.E.

Patented Shutters for Beams and Slabs.

SHUTTERS for beams comprise a series of metal side plates (H, J, K, M), *Fig.* 13, of various sizes, numbers of which are bolted together according to the size of the beam required, and a base-plate (D),



shown in cross section in *Fig.* 10, having longitudinally adjustable ends carrying vertically adjustable plates to provide for beams of different lengths and depths. The depth of the central plate (M) may be increased by a plate (N) for a large beam. The plates are connected at the bottom by bolts (B) on which rests the base-plate (D). The upper edges are held in the correct position by cramps (T) fitting in slots (P) in the plates and producing recesses in the top of the beam. A slab may be cast on plates (R), *Fig.* 11, supported by bars (S) resting in the recesses (Q) of the beams. The bars (S) and plates (R) are removed when the concrete has set. The slab and the beams may be cast by using the shutters of *Fig.* 13 with the bars (S) and plates (R). —British Patent No. 635,504. W. G. Shipwright. August 21, 1947.

Lifting Complete Floor Slabs.

In the September, 1952, number of the U.S.A. journal "Concrete" a method is described of raising concrete slabs precast at ground level to form floors or roofs at higher levels (*Fig.* 1). The method was invented independently by Mr. Philip Youtz and Mr. Tom Slick and developed by the Southwest Research Institute and the Institute of Inventive Research, both of San Antonio, Texas. The latter is also the agent for the licensing and operation of the system. In April, 1950, the first contract was carried out at Trinity University, San Antonio, and in 1951 over 1,000,000 sq. ft. of slabs were erected by this method.

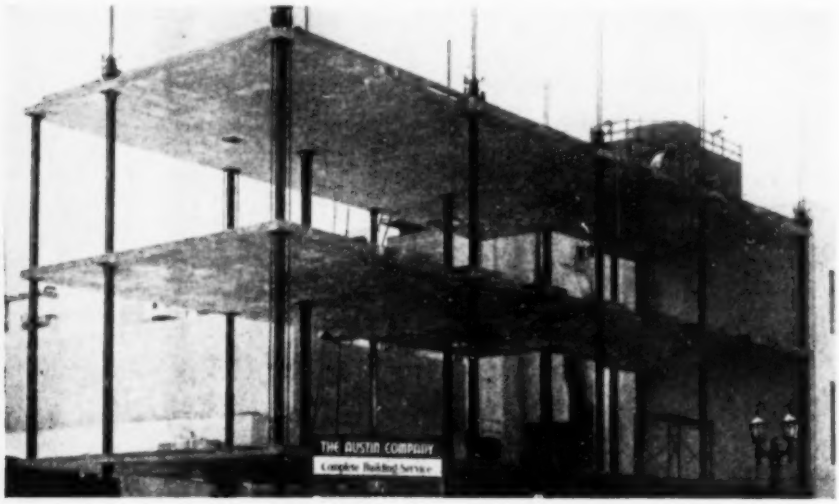


Fig. 1.—Floor and Roof Slabs in Position.

The system consists in casting the slabs one on top of another, and then lifting them to their final positions where plates embedded in the concrete slab are welded to steel columns.

The casting bed for all the upper slabs is the ground floor slab which can also be lifted to any desired height if its final position is not at ground level. When the fill is in place, reinforcement for the ground floor slab is fixed, and the conduits for mechanical and electrical services are inserted between the top and bottom layers of reinforcement. Shutters are fixed for the sides of the slab and for any openings in the floor, and the concrete is cast. The next slab is cast on this ground slab. The slabs are prevented from sticking to each other by paraffin wax or a wax-base liquid.

Lifting collars are placed around each column before fixing the reinforcement for the slabs.

Following an initial 3-in. lift and levelling at the columns, the slab is lifted to the desired height by hydraulic jacks placed on the top of each column head, the level of the slab being checked during lifting. Each jack contains a piston which pushes upwards two threaded rods without allowing them to turn and lifts the slab in steps of 3 in. The lower ends of the rods are connected to collars welded to plates embedded in the concrete (*Fig.* 2). The height gained at each stroke of the piston is maintained by nuts on the threaded rods which bear against the base of the jack. The nuts are tightened by chains actuated by small motors coupled to the movement of the jacks. At the end of every stroke of the piston the pressure is released and a spring pulls down the piston-head ready for the next lift of 3 in. Each jack is capable of

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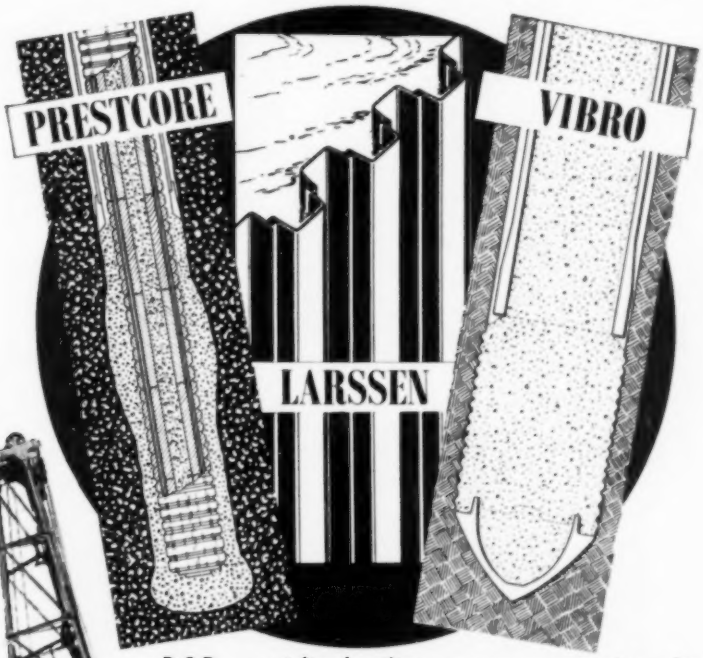

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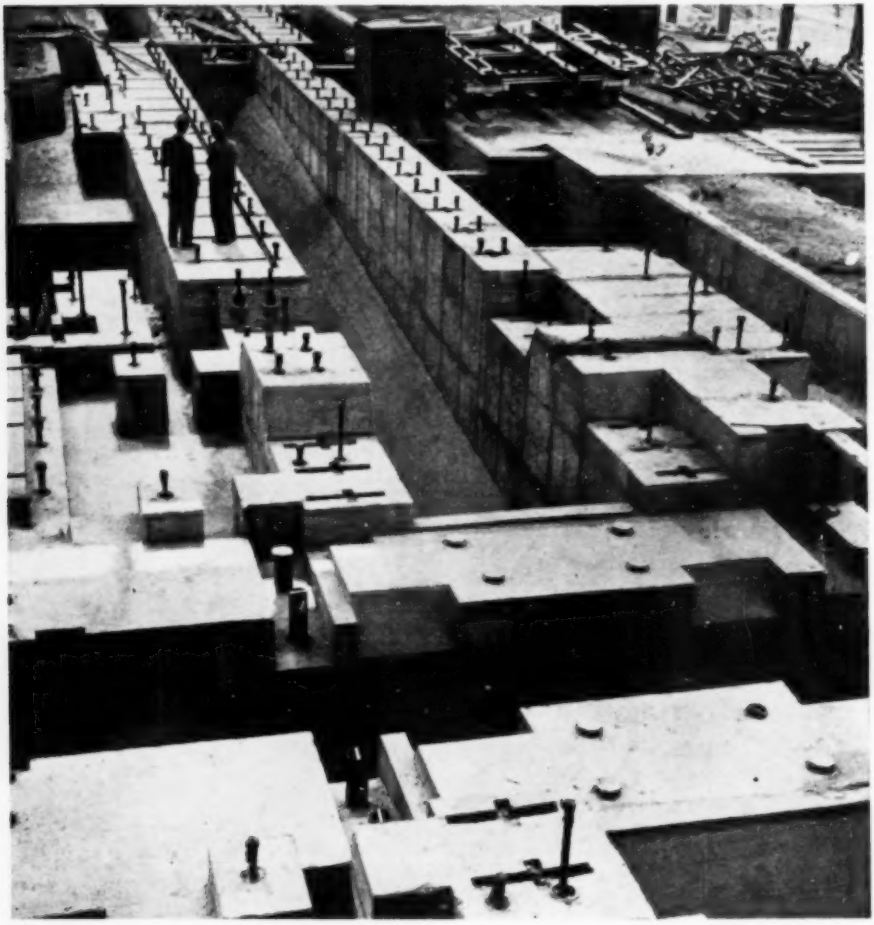

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lifting about 100,000 lb. At the desired height the slab is connected to the column by welding a collar on the column to the plate embedded in the slab. When the welds have cooled for 30 minutes, the slab is released from the lifting equipment. A crushing strength of 3000 lb. per square inch is required for the concrete and the slabs are allowed to cure for seven days to ten days.

The greatest height to which a slab has yet been lifted is 44 ft. during the construction of a two-story building in Los Angeles in which the second floor was at a height of 28 ft. and the first floor at 14 ft. Each slab is 80 ft. long by 30 ft. wide and is 8½ in. thick, the span between the columns being 18 ft. (*Fig.* 1). Because of their heights the roof and the second floor were lifted in two stages, the slabs being supported by timber props while the lifting equipment was raised for the second stage. The steel columns are hollow and were guyed and strutted during the erection of the slabs. The raising of the slabs was carried out by the Vagtborg Lift-Slab Corp. for the Austin Co. who were the general contractors. The largest slab lifted is the roof of an auditorium at Los Angeles; this slab weighs 530 tons, and was lifted by twelve jacks.

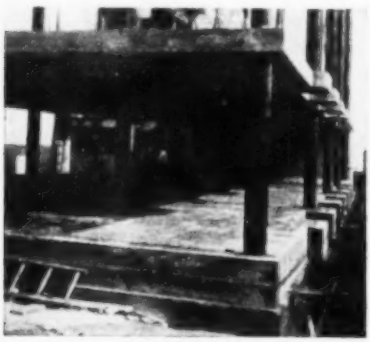

Fig. 2.—Raising the First Slab.

Fig. 3.—Collar for Connecting Slab to Column.

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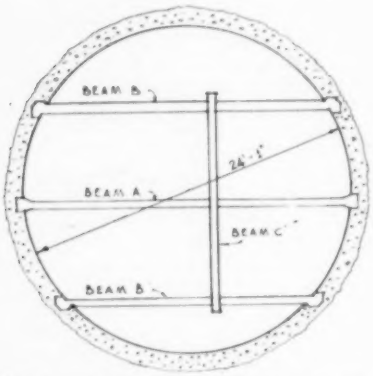
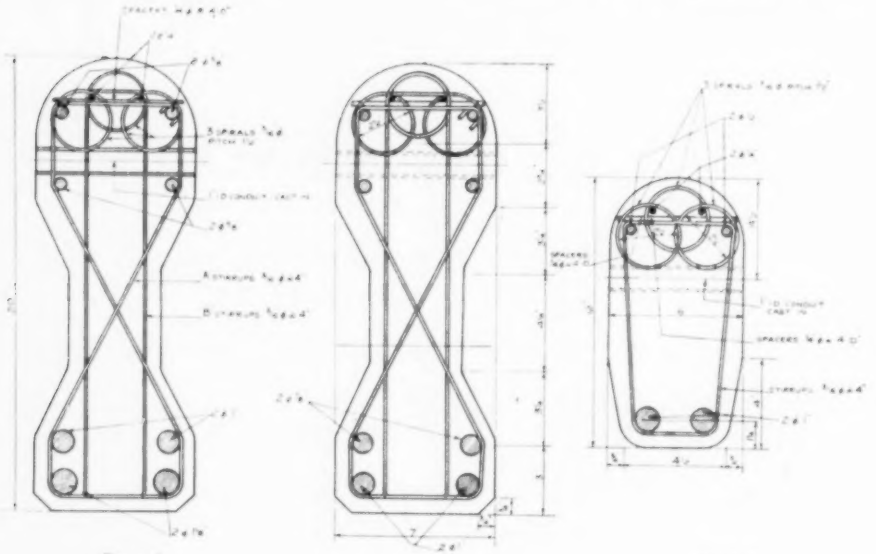


Fig. 1.—Arrangement of Beams in a Circular Shaft.

Eight beams similar to those shown in *Fig.* 2, but 19 in. deep by 6 in. wide, were made for testing. The concrete was a 1 : 1½ : 3 mixture with aggregate of a maximum size of ¾ in. and rapid-hardening Portland cement with an addition of 2 per cent. of calcium chloride by weight of cement. The water-cement ratio was 0·53 and the slump 3 in. The concrete was vacuum processed for twenty minutes and kept damp for seven days. Tests on 12-in. by 6-in. cylinders gave an average compressive strength of 7000 lb. per square inch at twenty-eight days. The working stresses were 1800 lb. per square inch in the concrete and 20,000 lb. per square inch in the steel.

The tests for resistance to impact were made on beams placed with their impact



Beam A. Beam B. Beam C.

Fig. 2.—Details of Beams.

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(*Continued on next page.*)

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(*Continued from previous page.*)

SITUATIONS VACANT. The British Reinforced Concrete Engineering Co., Ltd., require several qualified designers and detailers with specialist experience for their Stafford, Bristol, Glasgow, London, and Newcastle-upon-Tyne offices. Five-days' week and staff pension scheme. Apply to B.R.C. Engineering Co., Ltd., Stafford.

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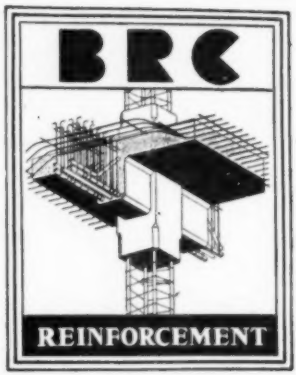

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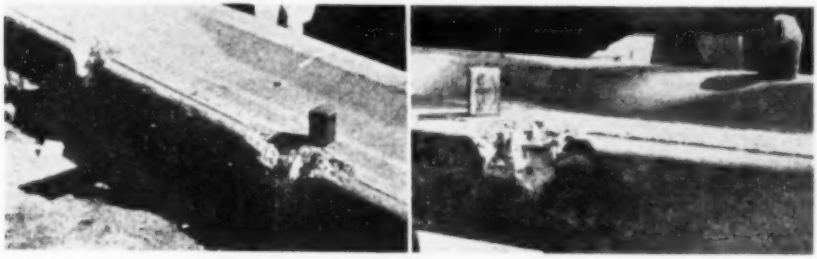

(*a*) Damage caused by 2¼ lb. missile. (*b*) Damage caused by 3¾-lb. missile.

Fig. 3.

faces (that is the upper faces) downwards and about 2 ft. above the ground. A grenade thrower was used to fire missiles weighing 2¼ lb. and 3¾ lb. at the flange of the beam at an average striking velocity of 120 ft. per second. The damage resulting from the first seven blows is shown in *Figs.* 3(a) and 3(b). When several missiles were fired at one spot the first generally produced most damage and subsequent blows caused little additional damage. It is considered that five blows with the 3¾-lb. missile produced a condition more severe than would be obtained in practice.

Bending and deflection tests were carried out on both damaged and undamaged beams, six beams being tested with the 19-in. dimension vertically and two beams on their sides. The beams were supported on rollers 24 ft. apart and the load applied equally at two points each 2 ft. from the centre of the span. The load causing failure of the beams in the upright position was between 26,000 lb. and 31,000 lb.; in the case of the beams on their sides the load causing failure was about 4000 lb. Damage to the concrete by impact did not appear to affect their load-carrying capacity. As a result of the tests the final design of three types of beam is as shown in *Fig.* 2. The foregoing is abstracted from an article by Mr. P. J. Doanides in the Journal of the American Concrete Institute for December, 1951.

A New Power Float.

THE machine shown in *Fig.* 1 is a new type of power float operated by an electric motor or a petrol engine. The disc is 2 ft. diameter and the machine weighs 200 lb. The advantages claimed for the machine are that a much drier mixture can be brought to a smooth surface without bringing an excess of cement to the surface, that the weight of the machine together with the action of the disc produces a surface which resists abrasion, and that it is quicker than trowelling by hand. By using a low water content it is possible to employ the float immediately after tamping and screeding. The machine is supplied by Columbus-Dixon, Ltd., of Wembley.

Fig. 1.

Roughening Concrete Surfaces.

THE machines shown in *Fig.* 1 are being used to prepare the soffits of the main access tunnels at London Airport to receive a finish of magnesite plaster. The work includes the levelling of the face of the concrete as left by the shutters and roughening the surface to form a key for the plaster. The total area to be treated is about 40,000 sq. yd. and is being done at a rate of about 1500 sq. yd. per week, each machine being capable of treating about 100 sq. yd. per day.

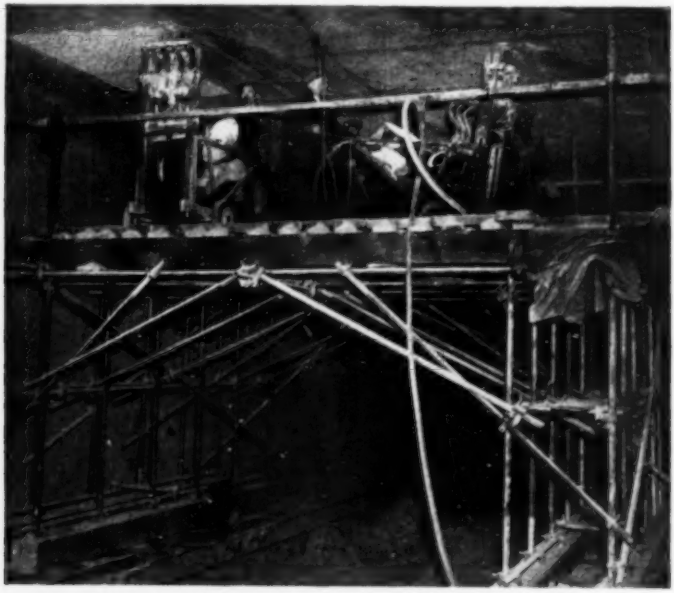

Fig. 1.—Roughening the Soffits of a Tunnel.

A description of these machines, which were originally developed for roughening concrete or stone road surfaces, was given in this journal for July, 1951. Since then the machine has been modified for work such as that described or for preparing floors for various finishes. The equipment comprises a compressor, hoses, and a roughening machine consisting of four rotary drills, with rose-shaped steel bits, carried in a portable cradle with two rubber-tyred wheels. The bits engage the surface of the concrete by being tilted forward by the operator who moves gradually forward. The depth of cut may be varied slightly (between $\frac{1}{16}$ in. and $\frac{1}{4}$ in.) by the operator. One pass of the machine roughens a strip about 2 ft. wide. Two men are required, one to operate the machine and the other to look after the compressor and assist the man using the machine.

The method is known as the "Met-texture" process and has been developed by the Metropolitan Construction Co., Ltd., who are sub-contractors of the Taylor-Woodrow Construction Co., Ltd., at London Airport.

Designing Welded Structures.

A BROCHURE entitled "Portal Frame Formulæ" giving simple expressions, based on the column-analogy method, for the determination of bending moments at the joints of single bay, single-story frames with hipped rafters and either pinned or fixed bases, and a brochure entitled "Welded Plate Girders," have recently been issued by the Quasi-Arc Co., Ltd., Bilston, Staffs., from whom copies may be obtained.

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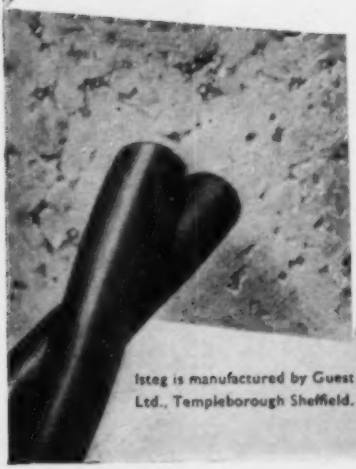

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